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ACI 45th Annual Convention, NEW YORK, February 23-25, 1949

JAN 4 1949

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

(ACI PROCEEDINGS Vol. 45)
PRINTED IN TWO PARTS—PART ONE

Vol. 20 **December 1948** **No. 4**

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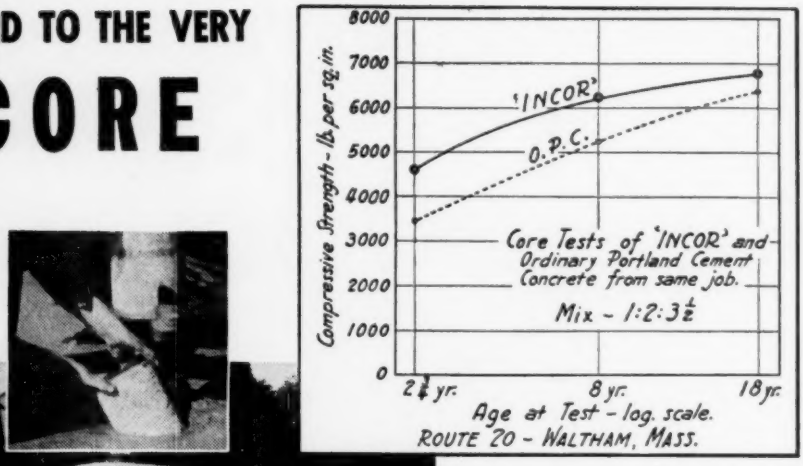

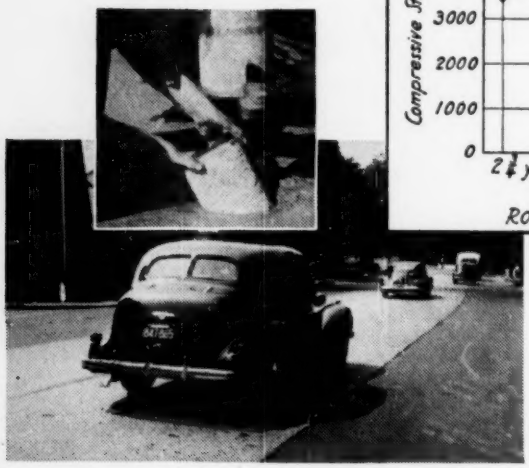

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A new method of designing flat slabs may prompt the revision of the present design methods specified in Sec. 709 of the ACI Building Code (ACI 318-47)

Rational Analysis and Design of Two-Way Concrete Slabs*

By C. P. SIESS† and N. M. NEWMARK‡

SYNOPSIS

A new method for the design of two-way building slabs is proposed and its step by step development is described. A new moment distribution procedure is used to compute moments in a number of rectangular slabs continuous over rigid beams. Several variables are studied in the analyses and include: the ratio of sides, the effect of discontinuous edges, the torsional stiffness of the beam, various types of loading, and combinations of panels of various sizes and shapes. Certain conclusions are stated regarding the types of loading to be considered and the values of beam torsional stiffness to be assumed in the development of the design procedure.

Moments obtained in the foregoing analyses are modified to take into account the effect of additional variables. First, the slab moments are increased by various amounts as a result of the deflection of the beams. Next, separate coefficients for dead and live load are replaced by combined coefficients for a live load-dead load ratio of 3.0. Then follows a study to determine the effect of discontinuous edges and this variable is eliminated from the procedure. Finally all slab moments are reduced by 20 percent in recognition of the redistribution of moments at high load. Additional studies are then made of the distribution of moments across the width of the slab and of the moments to be used for the design of the beams to complete the development of the design procedure.

A proposed design procedure is presented in specification form and is compared with other procedures of a similar nature and with the moment distribution procedure described in the appendix.

INTRODUCTION

This paper proposes a new method for the design of two-way building

*Received by the Institute Sept. 10, 1948. Scheduled to be presented in brief at the ACI 45th annual convention, New York, New York, February 24, 1949. Title No. 45-15 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE V. 20, No. 4, December 1948, *Proceedings* V. 45. Separate prints are available at 60 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than April 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

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slabs. The problem of development of the new design procedure has been attacked in three steps, as follows: First, a simple, approximate moment distribution procedure was developed for the calculation of moments in uniformly loaded continuous plates. Second, this procedure was used to study the moments in a number of arrangements of continuous slabs with the object of securing a better picture of the effects of the important design variables on the behavior of such structures. And finally, using the results from the previous step as a starting point, the various factors entering into the design of two-way slabs were studied and the design procedure was developed.

The development is described in sufficient detail to provide a proper understanding of the assumptions and philosophies of design on which it is based. An important feature of the new method is that the moments considered in analysis and design are the average moments acting on a section of the slab the full width of the panel. The method also differs from previous specifications in that corner and edge panels are designed for the same moments as interior panels. Other features include the consideration of the torsional stiffness of the supporting beams and of different types of live loadings.

The studies presented herein were entirely analytical in nature; no tests of two-way slabs were made.

THE DISTRIBUTION PROCEDURE

The distribution procedure is applicable to the calculation of moments in rectangular elastic plates supported on all four sides and continuous in two directions. Only loads distributed uniformly over an entire panel are considered, but all panels need not be loaded. The torsional rigidity of the supporting beams may be taken into account, but it is assumed in all cases that the beams do not deflect.

The procedure is one of "moment distribution" and is strictly analogous to the Hardy Cross* method for continuous beams and frames; that is, fixed-edge moments are calculated, unbalanced moments are distributed at a joint in proportion to the relative stiffness of the elements of the structure at the joint, and portions of the distributed moments are carried over to the other edges of the panel. The method as developed for plates, however, is not exact since the values of the stiffness and carry-over factors can be determined only approximately.

Since the moments on the edge of a continuous plate are not uniform but vary in some manner across the width of the plate, it is necessary to choose a particular value of moment to deal with in the distribution

*See for example: Cross, Hardy, "Analysis of Continuous Frames by Distributing Fixed-End Moments" *Trans. A.S.C.E.*, Vol. 96, 1932, pp. 1-156; or Cross, Hardy, and Morgan, Newlin D., "Continuous Frames of Reinforced Concrete," 1932, John Wiley and Sons, New York. Chap. IV.

procedure. Several possibilities present themselves: moment at the middle of an edge, maximum moment, total moment on the edge, or average moment on the edge. From the standpoint of physical interpretation the use of total moment is to be desired. However, use of the closely related average moment permitted certain simplifications of numerical constants and it was therefore chosen.

The distribution procedure was developed by making comparisons of exact solutions for moments in single rectangular plates supported on all sides and having various combinations of fixed and simply-supported edges. These exact solutions were obtained by means of the ordinary theory of flexure for plates, for a value of Poisson's ratio of zero.

The accuracy of the distribution procedure was determined by using it for the analysis of certain continuous slabs for which the correct moments were known. The error in the edge moments obtained by the approximate analysis was less than 6 percent, except for two cases in which the moments were quite small. The approximate positive moments varied from 8 percent less to 27 percent greater than the exact values.

The distribution procedure is described in detail in the Appendix.

ANALYSES OF CONTINUOUS SLABS

The relative ease with which moments may be computed by means of the distribution procedure made it feasible to undertake a large number of analyses of continuous slabs and thereby to investigate the effects of varying the proportions and other design conditions. The following factors were studied: (1) the ratio of sides of panels, (2) the presence of discontinuous edges, (3) the torsional restraint offered by the supporting beams, (4) various types of loading including all panels loaded and two types of partial loading, and (5) combinations of panels having various span lengths and ratios of sides. Eight slabs consisting of 25 similar panels arranged in five rows of five panels each and one slab consisting of 16 unequal panels were analyzed for various loading conditions. A total of 83 different analyses were made. Plans of the slabs studied are shown in Fig. 1. The dimensions of the panels and the intensity of loading were arbitrarily chosen so as to result in average moments that could easily be converted into coefficients of wb^2. For the slabs having 25 equal panels, $wb^2 = 10{,}000$.

The analyses of slabs with 25 equal panels will be discussed first.

Layout of slabs

The arrangement of panels shown in Fig. 1(a), (b), and (c) was chosen in order to obtain a variety of panel locations with respect to discontinuous edges. The center panel, I, is two rows distant from an edge and should be fairly representative of a typical interior panel.

Similarly a panel at the middle of a side of the slab, such as A or B, is two panels away from a corner and may be considered a typical edge panel. Panel C is a typical corner panel. Data may be obtained also for panels only one row removed from an edge or corner.

Ratio of sides

Under similar conditions of loading and restraint the moments in a given panel are dependent to a large extent on the ratio of sides, b/a. Three values of this variable were considered: 0.5, 0.8, and 1.0, as indicated in Fig. 1(a), (b), and (c), respectively. This range of ratios corresponds to that for which the distribution procedures was developed and for which it should give reliable results.

Torsional stiffness of beams

The effect of torsional stiffness of the supporting beams is to increase the restraint offered to rotation of the edges of a panel. This in turn results in a decrease in both positive and negative slab moments for an interior panel, but in an increase in negative moment on a discontinuous edge framing into a spandrel beam.

It was desired in these analyses to consider values of beam torsional stiffness that were representative of those in actual structures. In order to obtain such value, studies were made of typical two-way slab structures as described in two sources*. The torsional stiffness of the reinforced concrete supporting beams in these structures was computed by means of the following expression†

$$T_b = \frac{\pi^2 GJ}{b^2}$$

wherein T_b = torsional stiffness of beam having a span b,
G = modulus of elasticity in shear of the material in the beam,
J = measure of torsional rigidity of the cross-section of the beam.

The quantity J for a rectangular section having a width v and a depth d is given by Timoshenko‡ as:

$$J = \frac{v^3 d}{3}\left(1 - 0.63\,\frac{v}{d}\right)$$

A rectangular beam having a depth equal to the overall depth including the slab was considered in these calculations and the effect of T-beam action was ignored. Calculations based on the formula given by

*"Proposed Manual of Standard Practice for Detailing Reinforced Concrete Structures," American Concrete Institute, Detroit, 1946. See drawing 22 on page 41. (Now ACI Standard 315-48).
"Cost Estimates of Reinforced Concrete Floors," Portland Cement Association, Chicago, 1940.

†Newmark, N. M., "A Distribution Procedure for the Analysis of Slabs Continuous over Flexible Beams," Univ. of Illinois Eng. Exp. Sta. *Bul.* 304, 1938. See pp. 28-29.

‡"Theory of Elasticity," by S. Timoshenko, McGraw-Hill, New York, 1934. See the author's Eq. (155) on page 249.

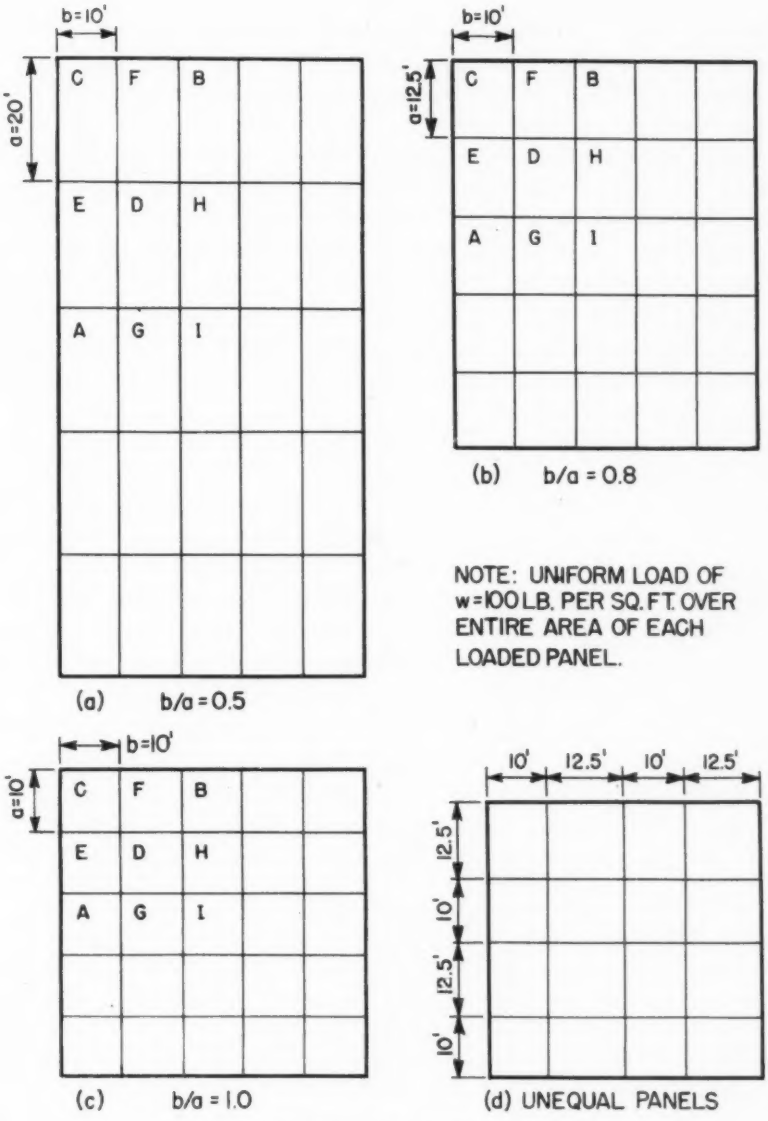


Fig. 1—Plans of slabs analyzed

Nylander* for the torsional rigidity of a T-section indicated that the stiffness of a T-beam having a flange width equal to one-quarter the span was only about 20 to 40 percent greater than that for the corresponding rectangular beam.

In the analysis of continuous slabs by the moment distribution procedure it is the ratio of the beam torsional stiffness, T, to the slab flexural stiffness K, that is of interest. Values of the slab stiffness were therefore determined in each case by means of the formulas developed for the distribution procedure, and values of the relative stiffness ratio, T/K, were computed for several panels in each of the typical structures. In obtaining this ratio, the stiffnesses of both the beams and the slab were based on the uncracked cross-section in accordance with the usual practice for reinforced concrete.

The range of relative stiffness, T/K, for all of the panels considered in the two typical structures was from 1.21 to 2.28, with an average of about 1.60. In the analyses of slabs with 25 equal panels, three values of T/K were considered: 0, 1, and 2. The zero value is of interest as a limiting case and also because design coefficients for two-way slabs have sometimes been based on the assumption that no torsional restraint is offered by the beams. The value of $T/K = 1$ was considered a minimum probable value for this ratio in an actual two-way slab supported on monolithically cast beams. Since all interior moments are decreased as the value of T/K is increased, a conservative value of this ratio must be used. The value of $T/K = 2$ was chosen as a reasonable maximum which must also be considered since the edge moment at a discontinuous edge exists only as a result of torsional restraint offered by the edge beam, and thus increases as T/K is increased.

Types of loading

Three types of loading were considered in these analyses, as follows:

(a) Uniform load over the entire area of the slab, referred to herein as "uniform loading". The dead load, or weight of the slab itself, is of this type and in some types of storage buildings the live load may correspond more closely to this loading than to either of the other types mentioned below. In the design procedures for flat slab floors specified in both the Joint Committee Report† and the ACI Building Code‡ the assumption of a uniform loading is implied by the nature of the expression used for total moment.

(b) Loading for maximum moments, commonly known as "checkerboard loading." The load is distributed uniformly over the entire

*Nylander, Henrik, "Torsion and Torsional Restraint of Concrete Structures", (In Swedish), Meddelanden, Statens Kommitté för Byggnadsforskning, Nr. 3, Stockholm, 1945, See Table 1 (c), p. 124.

†"Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete," Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete. Published by American Concrete Institute and others. June, 1940. See especially Section 835.

‡"Building Code Requirements for Reinforced Concrete (ACI 318-47)." American Concrete Institute, Detroit, 1947. See especially Section 1003.

area of each panel but only those panels are loaded that contribute an increase to the moment considered. For maximum positive moment in the interior of a panel and for maximum negative moment at the discontinuous edge of an edge or corner panel, alternate panels are loaded in checkerboard fashion. For maximum negative moment over an interior beam, the checkerboard pattern is applied on either side of and symmetrical about a line passing along the edge in question. Checkerboard loading has been used by Westergaard for the calculation of maximum moments in two-way slabs*. It is recommended for positive moments in both the Joint Committee Report, Sec. 803(a) (1), and the ACI Building Code, Sec. 702 (a) (2). It is also specified in the ACI Code, Sec. 1002(a) (6), for the design of flat slabs as continuous frames.

(c) The third type of loading is designated herein as "single-panel loading." For positive moment or for negative moment at a discontinuous edge, only the panel in question is loaded; for negative moment over an interior beam, only the two adjacent panels are loaded. The only precedent for this type of loading for positive moment is the Joint Committee Report, Sec. 806(b) (2), with reference to moments in one-way slabs. Its use for negative moments, however, is recommended throughout both the Joint Committee Report. Sec. 803(a) (2) and 806(b) (1), and the ACI Building Code, Sec. 702(a) (2) and 1002(a) (6).

It is obvious that the greatest moments will be produced by checkerboard loading and the least by uniform loading. However, if consideration is given in design to the redistribution of moments and stresses which may occur at loads near the ultimate, it will be found that the most critical loading is the one which covers the greatest area; namely, the uniform loading. Some consideration must also be given to the relative probabilities of occurrence of each of these patterns of loading. Obviously, the probability of obtaining a single-panel loading is very high, almost certain, since no more than two panels need be loaded. Next in order comes the uniform loading which is quite likely to occur in warehouses or similar structures if the aisle space is small or if the aisles are several panels apart. Even with wide or closely spaced aisles, it is possible to have several panels in the same row or in alternate rows fully loaded. The least probable of all the loading patterns considered is the checkerboard loading, which is necessary to produce absolute maximum moments. While a partial checkerboard loading might occur occasionally, the probability of exactly duplicating the entire pattern would seem to be small.

*Westergaard, H. M., "Formulas for the Design of Rectangular Floor Slabs and the Supporting Girders," ACI *Proceedings* V. 22, pp. 26-43, 1926.

Outline of analyses of slabs with equal panels

Three variables were considered in the analyses of slabs with 25 equal panels: (1) the ratio of sides, b/a; (2) the ratio of beam torsional stiffness to slab flexural stiffness, T/K: and (3) the type of loading. Three values each of the ratios b/a and T/K were combined as indicated in Table 1 for a total of eight different structures. The particular loadings used for each of these structures are also indicated in Table 1, and it may be noted that the analyses for the intermediate value of b/a were less extensive than those for the other values.

TABLE 1—SUMMARY OF TYPES OF LOADING CONSIDERED IN ANALYSES OF SLABS HAVING TWENTY-FIVE EQUAL PANELS

$\frac{T}{K}$	Values of b/a		
	0.5	0.8	1.0
0	Uniform Checkerboard Single-panel		Uniform Checkerboard Single-panel
1	Uniform Checkerboard Single-panel	Uniform Single-panel	Uniform Checkerboard Single-panel
2	Uniform Checkerboard* Single-panel*	Uniform Single-panel	Uniform Checkerboard Single-panel

*Loaded only for maximum edge moment at a discontinuous edge and maximum positive moments in edge panels.

All moments were computed by means of the distribution procedure described in the Appendix. For partial loadings, one panel was loaded at a time and the resulting moments on the edges of all panels were computed. Edge moments for a particular combination of loaded panels were obtained by superposition, and positive moments in the interior of the panels were computed from these values of the edge moments. The moments for uniform loading were obtained by direct calculation rather than by superposition.

For the structures having $T/K = 0$ the exterior edges were simply supported and use was made of modified distribution constants in order to simplify the calculations. Modified constants were also used in those cases for which the loading was symmetrical. Advantage was also taken of symmetry in reducing the number of panels in which moments were computed. For example, in the slabs having rectangular panels, moments were computed only for the panels designated by letters in Fig. 1(a) and (b). For the slab with square panels, moments were computed only for the panels labeled C, F, B, D, H, and I in Fig.

1(c). A total of 72 different analyses were made: 33 for $b/a = 0.5$ 11 for $b/a = 0.8$, and 28 for $b/a = 1.0$.

RESULTS OF ANALYSES OF SLABS WITH EQUAL PANELS

The moments obtained from the analyses described in the preceding paragraphs were compared in order to study the effects of the following three variables: (a) the location of a panel with respect to a discontinuous edge, (b) the type of loading, and (c) the torsional stiffness of the beams. The results of these studies are discussed in the following paragraphs.

Location of panel with respect to discontinuous edge

The general trend was for all moments except those at a discontinuous edge to increase as restraints against rotations of the edges were diminished. Thus the smallest moments were usually found for the interior panels, the next largest for an edge panel, with one discontinuous edge, and the greatest moments for a corner panel, with two discontinuous edges.

In none of the analyses were there any significant differences between the moments in any of the interior panels, whether one or two rows distant from the edge. In a square panel or in the long span of a rectangular panel the moments in an edge panel were increased only on sections perpendicular to a discontinuous edge. In the short span of a rectangular panel just the opposite was true; moments on a section parallel to a discontinuous edge were increased more than those on a section perpendicular to such an edge. The effects in corner panels, with two discontinuous edges, were approximately those that would be obtained by superposing the effects from each discontinuous edge considered separately.

The changes in moments due to a discontinuous edge were qualitatively the same for all of the structures analyzed. They differed in magnitude, however, as a function of the type of loading and the amount of beam torsional stiffness assumed. The greatest increases in moment were obtained for uniform loading of all panels, whereas the use of checkerboard loading, which minimizes the effects of continuity, resulted in a significant decrease in the differences between corner, edge, and interior panels. The results for single-panel loadings were intermediate to those for the other two types.

From the standpoint of beam torsional stiffness, the greatest effects of discontinuous edges were obtained for $T/K = 0$, as would be expected. If a value of $T/K = 1$, representative of a minimum amount of torsional stiffness was used, the differences between moments in panels at various locations became almost negligible for both the checker-

board and single-panel loadings, although fairly large differences still existed for the uniform loading. For $T/K = 1$, the maximum difference between moments in a corner and in an interior panel was 5 percent for checkerboard loading and 8 percent for single-panel loading. For $T/K = 2$, these differences were about one-half as great.

Type of loading

The greatest moments were produced by the checkerboard loading and the smallest by uniform loading. However, the difference decreased as the torsional stiffness of the beams was increased. For $T/K = 0$, both the positive and negative moments due to checkerboard loading were 3 to 19 percent greater than those for single-panel loading. For $T/K = 1$ the maximum difference was only 6 percent and the average about 3 percent, while for $T/K = 2$ the maximum was only 3 percent. Thus for a relatively small amount of beam torsional stiffness, the difference between moments for these two types of partial loading becomes negligible.

For $T/K = 1$ the moments due to uniform loading varied from 67 to 96 percent of those for single-panel loading; for higher values of torsional stiffness these percentage values were increased. In general, the differences between moments for uniform and either single-panel or checkerboard loadings were greatest for an interior panel and least for a corner panel.

Torsional stiffness of beams

As the torsional stiffness of the beams was increased, all moments except those at a discontinuous edge were decreased. The changes noted were greater for partial loadings than for uniform loading, and greater for corner or edge panels than for interior panels. An average decrease in moment of 18 percent was produced by a change from $T/K = 0$ to $T/K = 1$, and an additonal decrease of only 5 percent was produced by a further increase in torsional stiffness to $T/K = 2$. It would thus appear that a major portion of the effect of beam torsional stiffness on the interior moments is accounted for by the assumption of $T/K = 1$.

As the torsional stiffness of the beams was increased, the negative moments at a discontinuous edge were likewise increased. For a change from $T/K = 1$ to $T/K = 2$ this increase averaged 32 percent for uniform loading and 22 percent for the two partial loadings. The edge moments for $T/K = 2$ were on the average about 80 percent of those for a completely fixed edge, and it was estimated that a further increase to $T/K = 3$ would increase the edge moments only about 5 percent more.

Conclusions from analyses of slabs with equal panels

There are a great many variables which must be taken into account in the development of a design procedure for two-way slabs. However, unless a very elaborate method is desired, not all of them need appear directly in the final design procedure. It is usually possible to choose average or limiting values of some of the variables and to formulate a design procedure on the basis of these values without sacrificing too much in the way of accuracy. This procedure has been followed in this paper, and the following decisions are based on the results of the analyses of slabs with 25 equal panels.

(a) The ratio of sides, b/a, will be retained as a principal variable in the design procedure.

(b) The single-panel type of loading will be used for live load. This conclusion is based both on the results of the analyses and on the probability of occurrence of the two types of partial loading considered.

(c) Uniform loading of all panels will be used for dead load.

(d) For positive moments in all panels and for negative moments over interior beams a value of $T/K = 1$ will be assumed.

(e) For negative moments at an exterior edge a value of $T/K = 2$ will be assumed.

(f) The location of a panel, whether interior, edge, or corner, will be retained as a variable until after consideration has been given to the effects of beam deflections.

ANALYSES OF SLAB WITH UNEQUAL PANELS

The studies described in the preceding paragraphs were all concerned with slabs consisting solely of panels having the same size and shape. The analysis described herein was therefore undertaken in order to determine how the moments for a given panel are affected by the presence of adjacent panels of different sizes and shapes. The slab analyzed consisted of sixteen panels of three different sizes or shapes, as illustrated in Fig. 1(d). Torsional stiffness of the beams corresponding to a value of $T/K = 1$ was assumed throughout; the value of K used was the average for the two adjoining panels. The slab was analyzed for dead load as represented by uniform loading of all panels, and for live load as represented by single-panel loading. The two sets of moments thus obtained were then compared with moments obtained from the analyses of slabs with 25 equal panels by an approximate procedure described in the following paragraph.

For each of the unequal panels in the slab of Fig. 1(d) a corresponding panel having the same b/a and T/K ratio, occupying the same position relative to an edge or corner, and loaded in the same manner was chosen

from the slabs of Fig. 1(b) or (c). The moments in this panel, corrected where necessary for differences in span length, were then compared with those for the panel surrounded by unequal panels. For positive moments and for negative moments at a discontinuous edge the comparisons could be made directly. However, the negative moments over interior beams obtained from the equal panel solutions are different on the two sides of the beam because of the difference in size and shape of the two panels. These different moments were "balanced" by using the distribution procedure in a manner similar to that used in balancing fixed-edge moments. In this case, since $T/K = 1$ and since the slab stiffness K was practically the same for any two adjacent panels, the balancing procedure consisted simply of distributing two-thirds of the unbalanced moment equally to the two adjacent panels; that is, one-third to each panel and the remaining one-third to the beam.

Considering first only live load as represented by single-panel loading, it was found that the equal-panel moments determined and corrected as above agreed almost exactly with the moments obtained by actual analysis of the slab with unequal panels. Thus, the positive moments in all panels and the negative moments at a discontinuous edge are practically independent of size and shape of the adjacent panels, while the negative moments over interior beams can be determined very accurately from the solutions for equal panels by making the simple distribution described in the preceding paragraph. Although in this study the variation in span length of adjacent panels was only 25 percent, the very close agreement obtained suggests that much greater differences could exist without seriously affecting the accuracy of the results.

The agreement between the moments from the actual analysis of the slab with unequal panels and the approximate moments obtained from the equal-panel solutions was not as good for dead load as for live load. The maximum difference, however, did not exceed 8 percent and the approximate procedure may be considered satisfactory if the difference in adjacent span lengths does not exceed 25 percent.

The conclusions stated above regarding the use of the equal-panel solutions to obtain the moments in more irregular lay-outs furnish the justification for using the equal-panel analyses as the basis for a general design procedure.

EFFECT OF BEAM DEFLECTION ON MOMENTS IN SLAB

All of the moments thus far considered have been computed by means of the distribution procedure and therefore involve the assumption that the beams do not deflect. Actually, the supporting beams of a

two-way slab are not infinitely stiff and some deflection does occur. Its effect is to transfer moment from the beams to the slab and thus to increase the average moments in the slab. The exact amount of the increase depends on the actual stiffness of the beams relative to that of the slab, and may be different for positive and negative moments, for dead and live loads, for the long and short spans of rectangular panels, and for panels at various locations relative to a discontinuous edge.

The problem of determining the increase in slab moments due to deflection of the beams is a rather difficult one since exact solutions for moments in a slab continuous in two directions over flexible beams are not easily obtained. The procedure used herein to make such corrections is therefore approximate. It should be pointed out that in correcting the slab moments for the effect of beam deflection it was considered important to obtain proper relative values of the increase. That is, it was desired to obtain values of the increase in positive and negative moments, in the long and short spans, and for dead and live load which bore the proper relation to each other. Of course, the correct magnitudes of the corrections for each condition were also desired, and every attempt was made to secure them.

The procedure followed in determining the increase in slab moments due to beam deflections may be described generally as follows:

(a) Moments for the assumption of non-deflecting beams were obtained from the analyses of slabs with 25 equal panels described in the preceding sections. Moments were determined for typical corner, edge, and interior panels, for ratios of sides of 0.5 and 1.0 and for both dead and live load.

(b) Moments in the slab for zero beam stiffness were determined approximately by means of a modified frame analysis for each of the conditions of panel location, ratio of sides, and type of loading mentioned in the preceding paragraph.

(c) Typical designs of two-way slabs with reinforced concrete beams* were studied to determine the range of relative beam stiffness. In all cases, the stiffness of the beam relative to that of the slab was considered, and the gross uncracked cross-section was used in computing both stiffnesses. A T-beam section was considered for the beam in accordance with current practices. From these studies a minimum value of beam relative stiffness was selected for use in the remaining steps of the analysis.

(d) An approximate relation between moments in the slab and relative stiffness of the beams was then determined for each value of the ratio of sides.†

*The typical designs used in this study were the same as those used in the study of torsional stiffness. See footnote on page 276.

†Jensen, V. P., "Moments in Simple-Span Bridge Slabs with Stiffened Edges," Univ. of Ill. Eng. Exp. Sta. *Bul.* 315, 1939. See Table 1, page 87.

TABLE 2—INCREASE IN SLAB MOMENT DUE TO DEFLECTION OF BEAMS

Ratio of sides b/a	Moment	Average increase in slab moment due to beam deflection,* (percent)
0.5, short span	Negative Positive	5 7
1.0	Negative Positive	12 21
0.5, long span	Negative Positive	12 54

*The percentages shown were approximately the same for live and dead load and for all panels irrespective of their location with respect to a discontinuous edge.

(e) The relation from step (d) was used to interpolate between the limiting values of beam stiffness in step (a) and step (b) and thus to determine the increase in slab moment corresponding to the typical beam stiffness selected in step (c).

Although the increase in slab moment due to a change from infinitely stiff beams to beams of zero stiffness is quite large, the value of relative stiffness selected as a reasonable minimum for two-way slabs was of such a magnitude that not more than 5 to 10 percent of the total possible increase actually occurs as a result of the deflection of typical beams. The approximate increases in slab moments as determined by the procedure outlined in steps (a) through (d) are given in Table 2. As may be noted from the table, the magnitude of the increase in moment varied with the ratio of sides and span length and was greater for positive than for negative moments. There was practically no difference, however, between the increases for dead and live load moments or for the moments in typical corner, edge, and interior panels.

MOMENT COEFFICIENTS FOR COMBINED DEAD AND LIVE LOAD

Up to this stage in the development of a design procedure separate moments have been considered for dead and live loads. It is desired at this point, however, to make two changes. One is the transformation from actual moments as determined from the analyses of slabs of given dimensions and loadings to moment coefficients expressed in terms of wb^2, where w is the uniform load per unit of area on any panel, and b is the length of the shorter span. The other change involves the use of combined coefficients for dead and live load. It is of course possible to specify the use of separate coefficients for dead and live loads. However, if the values for these two cases are not too greatly different it is usually much more convenient in design to use a single set of coefficients which may be applied to the sum of the dead and live load.

After correction for the effects of beam deflections, the ratio of the coefficients for dead load to those for live load ranged from 0.72 to 0.94. These values of the ratio are sufficiently close to unity that combined moment coefficients computed for various values of the live load-dead load ratio should not differ too greatly. Consequently, values of combined coefficients corresponding to live load-dead load ratios of 1, 3 and 6 were computed for positive and negative moments in the long and short spans of typical corner, edge, and interior panels having ratios of sides of 0.5 and 1.0. From these calculations it was determined that coefficients for a live load-dead load ratio of 3 were not more than 3 percent in error on the unsafe side for a live load-dead load ratio of 6, and not more than 8 percent on the safe side for a ratio of 1. On the basis of these results the combined coefficients for a live load-dead load ratio of 3 were adopted for use in the design procedure.

EFFECT OF DISCONTINUOUS EDGES

A study was next made to determine whether the location of a panel with respect to a discontinuous edge could be eliminated as a variable in the design procedure. Combined coefficients for dead plus live load, corrected for the effects of beam deflection, were available for positive and negative moments in the long and short spans of panels having ratios of sides of 0.5 and 1.0. For each combination of these variables the coefficients for typical corner, edge, and interior panels were compared. It was found from these comparisons that the coefficients for an edge panel were from 2 to 10 percent greater than those for an interior panel, while the coefficients for a corner panel were 6 to 12 percent greater. The average increases over the coefficients for an interior panel were 4 percent for an edge panel and 8 percent for a corner panel. It would thus be feasible to specify for design purposes a set of basic moment coefficients for interior panels, and to use values 4 percent greater for edge panels and 8 percent greater for corner panels. It is believed, however, that the specification of different coefficients for interior, edge, and corner panels is a greater refinement than is consistent with the accuracy of the moment coefficients themselves. Therefore, it was decided to average the derived coefficients for typical corner and interior panels and to use these averages for all panels. The moments obtained in this manner are about 3 to 6 percent low for corner panels, 3 to 6 percent high for interior panels, and 2 to 4 percent low or high for edge panels, depending on the direction of the moment relative to an edge.

REDUCTION IN SLAB DESIGN MOMENTS BECAUSE OF REDISTRIBUTION

As the next step in the development of a design procedure a study was made of the phenomenon of redistribution of moments and stresses

in the slab at high loads with a view toward reducing the magnitude of the moment coefficients thus far obtained. The redistributions which may take place as the load on a two-way slab is increased are of four types, as follows:

(a) At low loads the moments on any given cross-section are non-uniformly distributed across the width of the slab, being greatest at the middle and smallest near the edges. As the load is increased and the middle portion of the width is overstressed, moment is transferred out toward the edges and the distribution becomes more uniform. This type of redistribution cannot be given any consideration in connection with a design procedure based on average moments over the entire width. Although the distribution may change, the average remains the same for this type of redistribution.

(b) If partial loadings are used for live load the maximum moments for which the slab is designed can not exist simultaneously at all sections. Thus, if two adjacent panels are loaded to produce maximum negative moment on one edge of a panel, the positive moment at midspan and also the negative moment on the opposite edge are both less than the maximum possible at those locations. Similarly, if one panel is loaded for maximum positive moment, the negative moments on the edges are less than the maximum moments obtained by loading two panels. Consequently, if the slab is loaded for maximum moment on a particular section and the load is increased until overstress occurs at that section, it is still possible for moment to be redistributed to other sections at which the slab is not so highly stressed. This is essentially a redistribution between positive and negative moments of the type that can also take place in a continuous beam. The capacity of the slab may be judged approximately by comparing the sum of the positive and negative moments provided for by the design with the sum of the moments actually produced by any given pattern of loading.

(c) For a given condition of loading the moments in a rectangular panel are greater in the short span than in the long span, because of the greater stiffness of the panel in the direction of the short span. However, as the load is increased and overstress occurs in the short span, the stiffness in that direction decreases and load is transferred to the long span. Thus, a deficiency in moment resistance in one direction may be offset at high loads by an excess in the other. This type of redistribution may occur also for square panels when two adjacent panels are loaded to produce maximum negative moment in one direction.

(d) The last type of redistribution to be considered is between the slab and the supporting beams and is the only one of the types discussed which is applicable only to two-way slabs and not also to flat

slabs. If the slab is overstressed in a given direction its stiffness is decreased and moment is transferred from the slab to the beams extending in the same direction. This type of redistribution can take place only if the slab in the direction at right angles has sufficient stiffness and resistance to effect the transfer of load laterally to the beams. No consideration was given in these studies to this type of redistribution as a basis for reducing the moments in the slab.

The reductions in slab moments proposed herein were based chiefly on the types of redistribution described in paragraphs (b) and (c). In each of these types redistribution is possible only because partial loadings are used for live load. Consequently, the limiting case of all panels loaded must also be considered. If a slab is designed for all panels loaded, and is then subjected to such a loading, the condition of overstress at one section or in one direction and understress in the other can not exist, and no redistribution of the type discussed in (b) and (c) can take place.

On the basis of extensive studies involving the moments due to all panels loaded as well as those due to partial loadings, a reduction of 20 percent in all of the slab moments was decided on. This reduction was chosen so that the moments in the slab for all panels loaded would not exceed the resistance provided by the design coefficients by more than a few percent. The following comparisons for a typical interior panel indicate the relation between the reduced design moments and the computed moments for the limiting case of all panels loaded. For a rectangular panel having $b/a = 0.5$, the individual moments for all panels loaded varied from 6 percent greater to 9 percent less than those provided by the reduced coefficients. The sum of positive and negative moments for all panels loaded was 4 percent greater for the short span but 5 percent less for the long span than the corresponding sum of the design moments. For a square panel the computed moments for all panels loaded exceeded the design moments by a maximum of 4 percent for a moment at a given section and by only 3 percent for the sum of the moments.

If partial loading is used for live load, the computed maximum moments in a typical interior panel for a live load-dead load ratio of 3 will exceed the proposed design moments by about 20 percent in all cases. However, for these conditions of partial loadings the sums of the computed positive and average negative moments is only from 7 percent greater to 5 percent less than the corresponding sums of the design moments.

It is believed that the reduction of 20 percent in slab moments is conservative. It is based almost entirely on the redistribution of moments that is possible if designs are based on maximum moments ob-

tained by partial loadings. At the same time, the proposed reduced moments agree quite well with the computed moments for loads in all panels, a condition which does not permit redistribution of the types considered herein. The magnitude of the reduction is less than the 28 percent proposed by Westergaard* based on the redistribution of moment in two-way slabs with all panels loaded. This reduction, however, was applied to moments resulting from partial loadings, and to the maximum moment on the cross-section rather than the average as herein. The choice of 28 percent was also influenced by the use of that figure in connection with flat slabs.

It is important that the relation between the 20 percent reduction made herein for two-way slabs and the 28 percent reduction for flat slabs be clearly understood. The reduction for two-way slabs is based on redistribution due to partial loadings; no appreciable reduction is made from the condition of all panels loaded. On the other hand, the 28 percent reduction for flat slabs is applied to the statical moment for all panels loaded and can be justified chiefly on the basis of loading test results or by the assumption that a smaller factor of safety may be used for that type of structure. If it is desired to extend this line of reasoning to two-way slabs, a type of structure containing even more redundant members than the flat slab, the starting point for any reduction would be the reduced moments given herein, and an additional cut of 28 percent or some such amount would be permissible. In this paper, however, the reduction has been limited to an amount which can be justified on the basis of the analyses and studies presented.

MOMENT COEFFICIENTS FOR VARIOUS RATIOS OF SIDES

In the preceding portions of this paper a set of design moment coefficients have been derived for two-way slabs having ratios of sides of 0.5 and 1.0. It was next necessary to determine additional values of the coefficients for the intermediate values of b/a. In order to interpolate between $b/a = 0.5$ and 1.0, use was made of the uncorrected moments obtained in the analyses of slabs having twenty-five equal panels for values of $b/a = 0.5$, 0.8, and 1.0. The variation of the design coefficients with b/a was assumed to be similar to that of these uncorrected moments. In the short span, the moments for $b/a = 0$ were taken equal to 80 percent of the corresponding moments in the central span of a five-span continuous beam loaded with the equivalent of single-panel loadings.

The final coefficients for positive and negative moments are presented in Fig. 2 as functions of the ratio of sides. Some modifications in the

*Westergaard, H. M., "Formulas for the Design of Rectangular Floor Slabs and the Supporting Girders," ACI, *Proceedings*, Vol. 22, 1926, pp. 26-43. See especially pages 31 and 32.

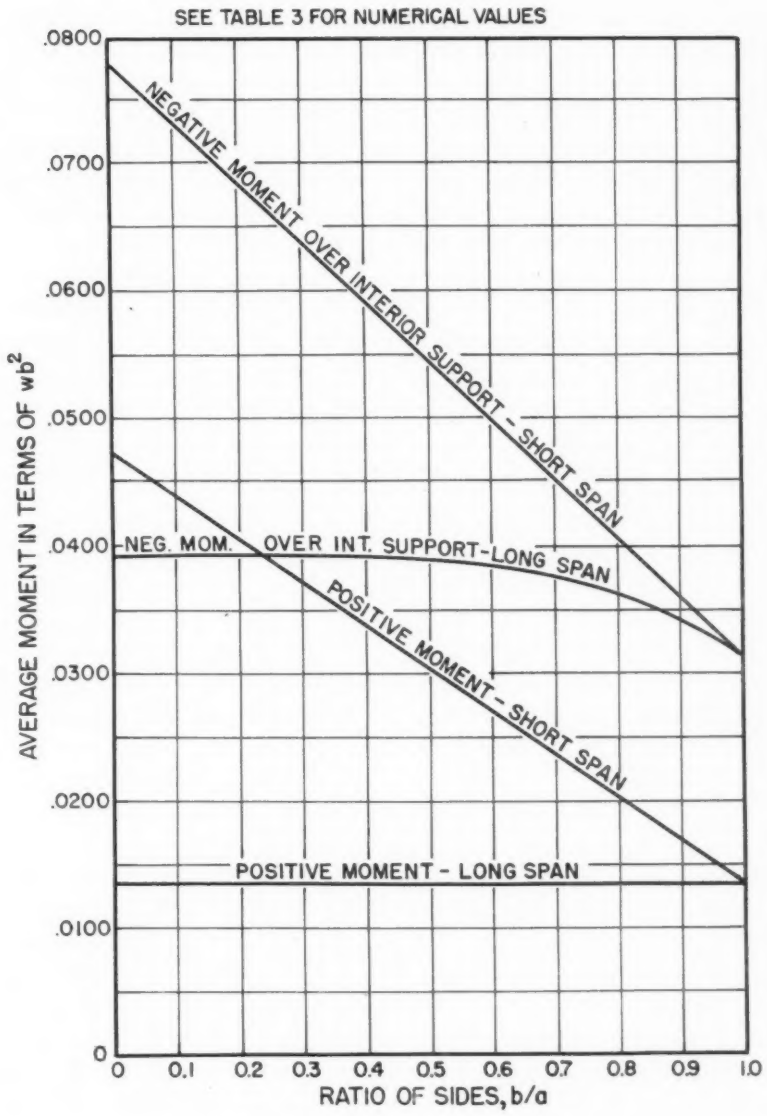


Fig. 2—Moment coefficients for two-way slabs

derived coefficients for positive moments were made in securing the curves shown in this figure. For example, the positive moment in the long span is shown equal to 0.0135 wb^2 for all values of b/a, whereas the computed coefficients were 0.0127 for $b/a = 0.5$ and 0.0134 for $b/a = 1.0$. Similarly, the computed coefficient for positive moment in the short span was 0.0290 and was increased to 0.0304 in the figure in order to obtain the somewhat simpler straight-line relationship. No significant modifications were made of the negative moments.

It should be noticed that the sum of the positive and negative moments in the short span for $b/a = 0$ is equal to 0.1252 wb^2, which may be compared with the total static moment in a beam: 0.1250 wb^2.

Coefficients are not given in Fig. 2 for the negative moments at a discontinuous edge since study showed that these moments could be expressed as a constant fraction of the negative moment over an interior beam. The following ratios of negative moment at a discontinuous edge to that over an interior beam were determined:

$b/a = 0$, short span: 0.656
$b/a = 0.5$, short span: 0.635
$b/a = 1.0$, both spans: 0.642
$b/a = 0.5$, long span: 0.615

It was decided, therefore, to specify for design a negative moment at a discontinuous edge equal to two-thirds the corresponding moment over an interior beam given in Fig. 2.

DISTRIBUTION OF MOMENT ACROSS WIDTH OF SLAB

The coefficients in Fig. 2 are for the average moments on a section extending the full width of the slab. Relatively little is known regarding the manner in which these moments are distributed across the width of the slab, except for a few exact solutions for structures which are to some extent different from a two-way reinforced concrete slab. These solutions were used, however, as a basis for estimating the distribution in two-way slabs, and the results are believed to be sufficiently accurate to be used in determining the manner of placing the slab reinforcement. Actually, if the total amount of reinforcement provided at a given section is sufficient to resist the total moment on that section, and if it is distributed in a manner even roughly approximating the distribution of the moment, the design will be satisfactory, since any differences in distribution of reinforcement and moment will decrease in importance as loads approaching the maximum capacity of the slab are reached.

A study was made of the distribution of moment on the edges and in the interior of uniformly loaded square and rectangular plates having all edges fixed. Since the beams were assumed to be non-deflecting in

these solutions and Poisson's ratio was taken equal to zero, the moments on each section varied from a maximum value at the middle of the width to zero at the edges. There was little difference between the distributions for positive and negative moments in the same span. In a square panel and in the long span of a rectangular panel the ratio of maximum to average moment was about 1.8. In the short span of a panel having $b/a = 0.5$ the ratio of maximum to average was about 1.5.

The effect of beam deflection is to make the distribution more uniform; that is, to decrease the ratio of maximum to average and to change the moment at an edge from zero to some finite value. These effects are greatest in the long span and least in the short span, as may be noted from the data in Table 2, and the deflection of the beams should therefore tend to make the distributions for long and short spans more nearly the same.

It was assumed as a result of these studies that the ratio of maximum to average moment for slabs on rigid beams would be decreased appreciably as the result of beam deflection and that the resulting ratios and distributions would be the same for both spans. On the basis of this assumption, and other studies of a qualitative nature, it was concluded that resistance should be provided for a moment equal to 1.25 times the average moment in the middle half of the width, and for a moment equal to 0.75 times the average in the outer quarters. The total moment thus provided for is equal to the total specified. The ratio of moment in the outer quarters or "column strips" to the moment in the middle half or "middle strip" is 0.60.

MOMENTS IN BEAMS

The studies and analyses described in this paper have been concerned almost exclusively with moments in the slab; no direct calculations have been made for moments in the beams. It is essential, however, in formulating a design procedure that moments in the beams be considered, and that values be specified for use in design which are consistent with those recommended for the slab.

The only basis for obtaining moments in the beams from the analyses described herein is provided by the condition that the sum of the moments in the beams and in the slab must equal the static moment for the loading considered. Since the slab moments are known for a number of cases, the total beam moments may be determined. However, this calculation gives no indication of the manner in which the beam moments are, or should be, distributed between the sections for positive and negative moment.

A study of exact solutions for uniformly loaded rectangular plates having fixed edges, and other solutions for uniformly loaded plates

continuous over rigid beams, indicated that the total moment carried by the beams is very nearly equal to that obtained on the basis of the load distribution specified in the ACI Code, Sec. 709, Method 2, Par. (d). The section referred to specifies that

> The loads on the supporting beams for a two-way rectangular panel may be assumed as the load within the tributary areas of the panel bounded by the intersection of 45-degree lines from the corners with the median line of the panel parallel to the long side.

It is permissible under this specification to replace the triangular and trapezoidal load distributions thus obtained by equivalent uniform loads as given by the following equations:

$$\text{For the short span: } q_b = \frac{wb}{3} \quad \ldots\ldots\ldots\

such as a single span or alternate spans loaded for maximum positive moment and adjacent spans loaded for maximum negative moment, the resistances provided will be such that if the beam is loaded for maximum effect at one section, it will be understressed at other sections. Similarly, if the various panels of the slab are loaded so as to produce maximum moments at a given section of the beam, some of the moments produced in the slab will be less than those for which it was designed. Also, the beam at the opposite edge of a panel adjacent to the beam in question will be understressed by a relatively large amount. Consequently, load will be shifted from the highly stressed beams to the understressed slab and opposite beam.

The factors just discussed are believed to warrant a further reduction in the moments for which the beams are designed. In order to determine just how much of a reduction should be made the behavior of the structure with all panels loaded was considered. For this condition, no redistribution can take place and the criterion used was that the sum of the moments resisted by the beams and by the slab should not be less than the static moment for a uniform load over all panels. It was found from several studies that if the beams were designed for an equivalent uniform load equal to 80 percent of that given by equations (1) and (2), and if the moment coefficients specified in the ACI Code, Sec. 701 (c), are used, the resulting moments will satisfy substantially the criterion just stated. For example, the sums of the slab and beam moments provided for in a typical interior panel designed for the conditions laid down above are as follows:

$b/a = 0.5$, short span: $0.1261\ wab^2$
$b/a = 0.5$, long span: $0.1256\ wba^2$
$b/a = 1.0$, both spans: $0.1268\ wb^2$

The units have been chosen for each case so that the corresponding static moment coefficient is 0.1250.

It should be understood that the recommended procedure for designing the beams is not equivalent to designing them for all spans loaded. In that case, the loads given by equations (1) and (2), reduced by not more than 10 percent for the effect of deflection, would be used, and the moment coefficients for positive and negative moments in an interior panel would be taken equal to 1/24 and 1/12, respectively. However, if partial loadings should then occur, the beams designed on this basis would be greatly overstressed in positive moment and only slightly overstressed in negative moment. In the procedure proposed herein the moment is divided between the sections for positive and negative moment in proportion to the maximum moments at each section obtained with partial loadings. Thus, for such loadings, the overstress will be less than 20 percent for both positive and negative moments,

while for all spans loaded there will be an overstress of less than 15 percent for negative moment and an understress of about 20 percent for positive moment. The philosophy underlying the recommended design procedure is that some overstress may be permitted at one section if the additional resistance needed is available at another section or in another element of the structure.

PROPOSED DESIGN PROCEDURE FOR TWO-WAY SLABS

The procedure proposed in the following praagraphs is based almost entirely on the studies and analyses presented in this paper. The form of presentation is patterned after that of the 1940 Joint Committee Report, Sec. 809 through 815.

(a) General

These recommendations are based on analyses of continuous slabs made by means of the distribution procedure described herein. The following loadings have been considered in determining the maximum moments due to live loads: (1) a single panel loaded for maximum positive moment and for maximum negative moment at a discontinuous edge, and (2) adjacent panels loaded for maximum negative moment over an interior beam. All panels were considered loaded for the computation of moments due to dead load.

The following important assumptions have been made in the development of this procedure:

(1) In the computation of positive moments and of negative moments over interior beams, it has been assumed that the beams possess torsional rigidity equivalent to the flexural stiffness of the slab in the adjacent panels. In the computation of negative moments at a discontinuous edge, the edge beams were assumed to offer torsional restraint twice as great as that assumed above.

(2) The effects of deflection of the beams on both the magnitude and the distribution of the moments in the slab have been considered.

(3) All moments determined for the slab have been reduced by 20 percent in recognition of the redistribution of moment which can take place at high loads.

(4) Moment coefficients for the slab are based on a ratio of live load to dead load of 3.0.

(b) Limitations

The procedure proposed herein is intended to apply only to reinforced concrete slabs consisting of rectangular panels continuous in two directions and supported on all four sides by reinforced concrete beams built monolithically with the slab, or by steel beams encased

in concrete. The slab should contain three or more rows of panels in each direction, the ratio of width to length of any panel should not be less than 0.5, and adjacent span lengths should not differ by more than 25 percent.*

Although the ratio of live load to dead load assumed in the derivation of the moment coefficients was 3.0, the procedure may be used with safety and economy for values of this ratio varying between 1.0 and 6.0.

This procedure is applicable only to those cases in which the live load is distributed uniformly over the entire area of each loaded panel; partial loadings within a panel and concentrated loads are not considered.

(c) Design sections

The critical section for negative moment should be taken along the edges of the panel at the faces of the supporting beams.

The critical sections for positive moment should be taken along the center lines of the panels.

For placement of the reinforcement, the width of the slab at each critical section should be divided into a middle strip, covering the central half of the width, and two column strips, covering the outer quarters.

For bending up truss or bent bars used for positive moment reinforcement, and for the calculation of anchorage requirements for straight bars similarly used, the lines of contraflexure for positive moment should be assumed to be at a distance of $\frac{1}{6}$ the shorter span from each edge of the panel.

For computing anchorage requirements for negative moment reinforcement, the lines of contraflexure should be assumed to be at a distance of $\frac{1}{5}$ the shorter span from each edge of the panel.

(d) Bending moment coefficients

Coefficients for average bending moments for various values of b/a are given in Table 3. These coefficients when multiplied by wb^2 give the average bending moment per unit of width of slab. The values given apply to all panels, whether located in the interior, at an edge, or at a corner. Negative moments at an exterior or discontinuous edge should be taken as two-thirds the moments over an interior beam.

Interpolation between the values of b/a given in the table is permissible.

In the middle strip, a moment equal to 1.25 times the average moment should be used, and in the column strips, a moment equal to 0.75 times the average should be used.

*When these limitations are exceeded it is recommended that moments in the slab be computed by means of a rational analysis using the distribution procedure described in the Appendix. If this is done the beams should be designed to resist that portion of the total static moment not carried by the slab.

TABLE 3—MOMENT COEFFICIENTS FOR TWO-WAY SLABS

Values are average moment in terms of wb^2. See Fig. 2 for plot of coefficients vs. b/a.

Moment	Span	Ratio of sides b/a						
		0	0.5	0.6	0.7	0.8	0.9	1.0
Negative moment over interior beams	Short	0.0780	0.0548	0.0501	0.0455	0.0408	0.0362	0.0315
	Long	0.0390	0.0390	0.0387	0.0378	0.0363	0.0342	0.0315
Positive moment	Short	0.0472	0.0304	0.0270	0.0236	0.0202	0.0169	0.0135
	Long	0.0135	0.0135	0.0135	0.0135	0.0135	0.0135	0.0135
Negative moment at exterior edge	To be taken equal to two-thirds the coefficient for negative moment over an interior beam.							

If desired in connection with the spacing of reinforcement, any reasonable variation of moment across the middle and column strips may be assumed provided the following conditions are met: (1) The moment is least at the edge of the panel and increases toward the center. (2) The moment at the edge of the panel is assumed not less than one-half the specified average value from Table 3. (3) The total moment provided for in each strip is not less than that specified herein.

(e) Distribution of unequal negative moment at supports

In applying the moment coefficients for negative moment over interior beams from Table 3 to adjacent panels of varying dimensions or unequal loading, the negative moments on either side of a supporting beam may differ materially. When this condition exists some modification of the moments should be made based on the relative rigidity of the slabs and the torsional restraint offered by the beam. For this purpose, it is assumed that the supporting beams offer a restraint equivalent to the average of the stiffnesses of the adjacent slabs. On this basis two-thirds of the unbalanced negative moment should be distributed to the two spans in proportion to their respective stiffnesses. If the thickness of the slabs is the same for the two panels, and if their span lengths do not differ by more than 25 percent, it will be satisfactory to assume that their stiffnesses are equal.

Because the conditions of loading considered for positive moments are different from those for negative moments, adjustments of the type described above have no effect on the positive moments obtained from the coefficients in Table 3.

(f) Bending moments and shears in supporting beams

The loads on the supporting beams for a two-way rectangular panel may be assumed as the uniformly distributed load within the tributary areas of the panel bounded by the intersection of 45-degree lines from the corners with the median line of the panel parallel to the long side.

The distribution stated above may be transformed to equivalent uniform loads per unit length of beam, due to one loaded panel only, giving the same maximum moment in a simple beam, as follows:

$$\text{For the short span, } = \frac{wb}{3}$$

$$\text{For the long span, } = \frac{wb}{3} \cdot \frac{3 - (b/a)^2}{2}$$

wherein w is the uniform load per unit of area on the slab, b is the length of the short span, and a is the length of the long span.

Moments and shears in the beams should be computed for 80 percent of the equivalent uniform load given above, using the coefficients for maximum moments and shears given in the ACI Building Code (ACI 318-47), Sec. 701(c).

(g) Shear in slab

Shearing stresses in the slab should be computed on the assumption that the load is distributed to the supporting beams in accordance with (f) above.

COMPARISON OF DESIGN PROCEDURES

A comparison of the proposed design moments in Table 3 with those from other procedures is of value in illustrating the significance of the various assumptions on which the new procedure is based. In Fig. 3 the average design moments from Table 3 and Fig. 2 are compared with corresponding average moments for a typical interior panel based on the procedure proposed by Westergaard, and that specified in the 1940 Joint Committee Report, Sec. 811, and in the ACI Code (ACI 318-47) Sec. 709, Method 2. The bases for computing the average moments given in Fig. 3 were as follows:

Westergaard procedure

Values of moments for a typical interior panel were computed from the expressions given in Fig. 5 of the paper referred to. Since separate expressions are given for the design moments in the middle and side strips, average moments were computed on the basis of the relative widths of these strips. In this method, the width of each strip is one-fourth the shorter span.

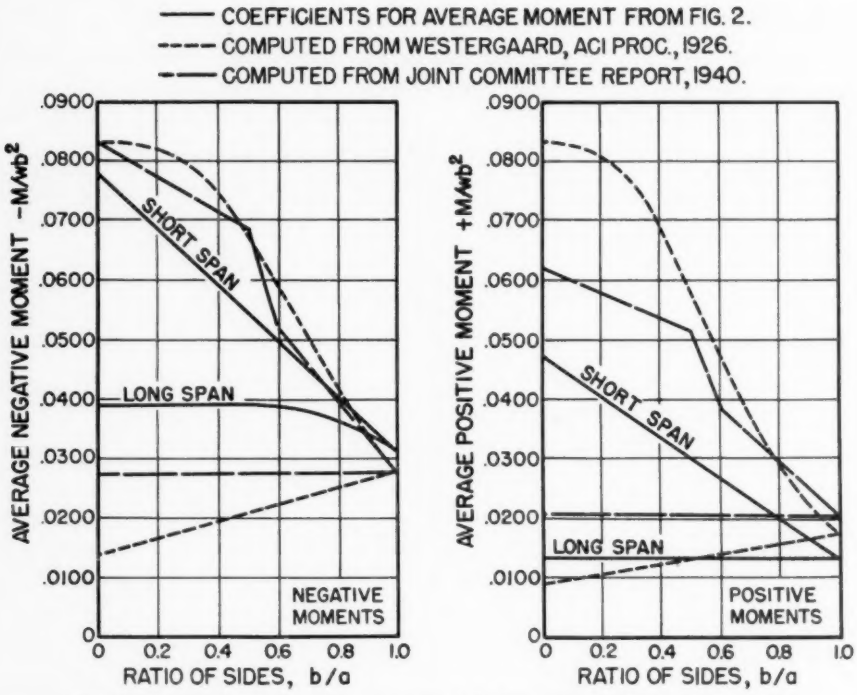


Fig. 3—Comparisons of various proposed design moments for two-way slabs

Joint Committee Report—ACI Method 2

Moment coefficients were taken from Table 5 of the J. C. Report (Table 3 of ACI Method 2) for "Case 1—Interior Panels." These values apply for the middle strip, and moments two-thirds as great are specified for the column strips. For b/a greater than 0.5 the width of each column strip is one-fourth the total width of the section being considered. For b/a less than 0.5 the width of each column strip for moment in the short span is equal to one-half the shorter span. These relations permitted the calculation of average moments from the tabulated coefficients.

It may be noted in Fig. 3 that negative moments in the short span are not greatly different for the three procedures but in the long span the negative moments proposed herein are considerably greater than those for either of the other methods. On the other hand, positive moments in both spans were in general appreciably less for the procedure given in this paper. The smaller positive moments are particularly interesting in view of the fact that the distribution procedure on which the design procedure is based nearly always gives positive moments that are greater than the correct values. For example, in a square interior panel on rigid beams the positive moment computed

by means of the distribution procedure for all panels loaded was 25 percent greater than the correct moment. In addition, it may be noted in Table 2 that the increases in slab moment due to deflection of the beams were significantly greater for positive than for negative moments. Consequently, it would seem that the proposed positive moments are conservative.

The differences between the moments derived in this paper and those proposed by Westergaard may be explained by the following differences in the assumptions made in the derivations:

(a) Torsional rigidity of the beams was considered herein, but not by Westergaard.

(b) The moments in this paper are based on a combination of single-panel and uniform loading, while Westergaard's values are based on checkerboard loadings.

(c) The slab moments are increased herein to take account of the effects of beam deflection, while Westergaard considered these effects only as they affected the distribution of moment across the width of the slab.

(d) The reduction of 20 percent made herein for the effects of redistribution is less than the reduction of 28 percent made by Westergaard.

The assumption of torsional beam stiffness probably accounts for a good share of the decrease in positive moments. The corrections for deflection of the beams and the reduction due to redistribution are opposite in sign and tend to counterbalance each other, but only partially since the increase for beam deflection was greater for positive than for negative moment while the decrease for redistribution was the same for all cases. The difference in loading would tend to account for a general decrease from Westergaard's values.

The design procedure for two-way slabs as specified in the J. C. Report and in the ACI Code, Method 2 is supposedly based on the moments proposed by Westergaard. It is obvious, however, from the values plotted in Fig. 3 that some modifications have been made, and since their nature is not known, no comparisons of assumptions can be made other than those already made for the Westergaard procedure.

It has been recommended that the proposed design procedure be used in certain regular cases, and that the distribution procedure be used in those cases falling beyond the specified limits. It is therefore of interest to compare the moments obtained by these two procedures. The principal quantitative differences between the moments obtained by the distribution procedure and those given in Table 3 are due to two corrections: an increase due to deflection of the beams and a 20 percent decrease because of redistribution. Examination of Table 2

shows that except for positive moments in the long span, the increase due to beam deflection is always less than 25 percent. Consequently, it would be expected that, with the exception noted, the moments obtained by means of the distribution procedure would be greater than those in Table 3, assuming of course that the same torsional stiffness, the same types of loading, and the same live load-dead load ratios are used in both cases. A few studies involving panels having ratios of sides between 0.5 and 1.0 confirmed this fact. The negative moments obtained by means of the distribution procedure were individually as much as 20 percent greater than those computed from the design procedure and Table 3. On the other hand, positive moments in the short span were from 5 percent smaller to 15 percent greater, and those in the long span were as much as 24 percent smaller for the distribution procedure than for the design procedure. Some of the difference for positive moments in the long span is due to the fact that the design moments in Table 3 and Fig. 2 have been considered constant instead of decreasing as b/a decreases, as was actually the case. Thus, the error is not as great as the above comparison would indicate. Nevertheless, there is a real error since the increase in positive moment due to beam deflection in the long span is considerably greater than the subsequent reduction because of redistribution. This error is not too important, however, since the moments in question are usually quite small and are more or less secondary in nature for the spans of small b/a for which the error is greatest.

All of the errors discussed above tended to increase in magnitude as the value of b/a became smaller, and it is likely that even greater errors than those mentioned would exist for very small values of b/a.

It is evident from the foregoing remarks that a design based on moments obtained by means of the distribution procedure would in general be more conservative than one based on the moments in Table 3. Since the distribution procedure is recommended only for special or extreme cases in which increased conservatism is usually desirable, the use of the moments thus obtained without further correction can be justified. On the other hand, it is possible for the designer to take into account, however crudely, the effects of beam deflection and also to make a reduction for redistribution, provided that care is taken to see that the conditions of statics are satisfied for all panels loaded, taking into account the moments for which the beams are designed.

SUMMARY

The step-by-step development of a new design procedure for two-way building floor slabs has been described. A new moment distribution procedure was used to compute moments in a number of rectangular

slabs continuous over rigid beams. The several variables studied in these analyses included: the ratio of sides, the effect of discontinuous edges, the torsional stiffness of the beams, various types of loading, and combinations of panels of various sizes and shapes. From these studies certain conclusions were reached regarding the types of loading to be considered and the values of beam torsional stiffness to be assumed in the development of the design procedure.

The moments obtained in the foregoing analyses were then modified to take into account the effects of additional variables. First, the slab moments were increased by various amounts as a result of the deflection of the beams. Next the separate coefficients for dead and live load were replaced by combined coefficients for a live load-dead load ratio of 3.0. A study was then made to determine the effect of discontinuous edges and this variable was eliminated from the procedure. Finally all slab moments were reduced by 20 percent in recognition of the redistribution of moments at high loads. Additional studies were then made of the distribution of moments across the width of the slab, and of the moments to be used for the design of the beams, in order to complete the development of the design procedure.

The proposed design procedure is presented in specification form, and is compared with other procedures of a similar nature and with the moment distribution procedure described in the Appendix.

ACKNOWLEDGMENT

This paper is based upon a thesis by C. P. Siess submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy in Engineering in the Graduate College of the University of Illinois, 1948. The investigations for this thesis were carried out under the direction of N. M. Newmark.

APPENDIX—A DISTRIBUTION PROCEDURE FOR THE COMPUTATION OF MOMENTS IN PLATES CONTINUOUS IN TWO DIRECTIONS

General description

The distribution procedure is applicable to the calculation of moments in rectangular elastic plates supported on all four sides and continuous in two directions. Only loads distributed uniformly over an entire panel may be considered, but all panels need not be loaded. The torsional rigidity of the supporting beams may be taken into account, but it is assumed in all cases that the beams do not deflect. It is further assumed that Poisson's ratio is equal to zero.

The prodecure is analogous to the Hardy Cross moment distribution method for continuous beams and frames; that is, fixed-edge moments are calculated, unbalanced moments are distributed in proportion to the relative stiffnesses of the elements of the structure, and portions of the distributed moments on each edge are carried over to the other edges. The first step is the determination of the average moments on the edges of each loaded panel, for all edges considered fixed. Coefficients for these moments, as well as numerical values of the other distribution constants, are given in the following section.

In general, the fixed-edge moments at the junction of two panels of the slab along the line of a support will be different for the two panels; that is, the moments at an edge will not be statically balanced. The unbalanced moment, equal to the algebraic difference in moments for the two panels, is distributed to the two panels in such a manner as to equalize the moments on the two sides of the edge, both in magnitude and in sign. If torsional rigidity of the supporting beams is considered, the moments in the beams may be handled in the same way as moments in columns of a continuous frame. The unbalanced moments at each edge are distributed to the adjacent penals in proportion to the relative values of the panel *stiffness factors*, K, as defined in the following section.

The distribution of unbalanced moments may be visualized as the releasing of restraints on an edge which was previously held fixed. Distribution to the various elements in proportion to their stiffnesses satisfies the condition that the slopes resulting from the distributed moment must be equal on both sides of a "joint." In this method, it is the average slopes which are made equal, and therein lies one of the basic approximations of the distribution procedure.

The redistribution of moments occuring when one edge of a panel is released introduces additional unbalanced moments at the other edges of that panel. These moments are said to be "carried over" from the released edge, and their magnitude is determined as the product of the distributed moment and certain *carry-over factors*, C, which are functions of the ratio of sides, b/a, for the panel. Four carry-over factors are needed, two for the long edges and two for the short edges of a panel; one of each pair determines the moment carried over to an opposite edge, the other the moment to adjacent edges. Numerical values of these constants are given in the following section.

As in the Cross method, the two operations of distributing moments and carrying over are performed at each joint in turn, or at all joints simultaneously, as the analyst prefers, and are then repeated as many times as necessary until the moments on the two sides of an edge are balanced to the degree of accuracy desired. When this has been accomplished, the average moments on the edges of the continuous panels may be determined by adding algebraically the fixed-edge moments, the distributed moments, and the carried-over moments at each location.

The positive moments in the interior of a panel may be obtained as the summation of the following quantities:

(1) the moments due to the load acting on the panel simply supported on all edges,

(2) the moments produced in the interior of the panel by the moments acting on each edge.

Numerical values of the moments in a simply-supported slab with uniform load are given in the following section. The average moments produced in each direction by the edge moments are determined for each edge and for interior moments in each direction by means of *positive moment correction factors*, F, as described subsequently. The results given by this method of computing the positive moments in a panel are not exact, but it is believed that the values obtained are conservative in all cases, without being too far on the side of safety. The principal reason for the relative crudeness of this calculation is that the location of the section on which the maximum positive moment occurs varies considerably with the edge conditions of a panel. Consequently, it has been necessary to choose values of the positive moment correction factors, F, which will give a reasonably correct answer for any condition.

Numerical values of constants

Numerical values of the moment coefficients and distribution constants are given in this section. The identifying subscripts a and b, refer at all times to an edge or to a section parallel to an edge having a length of a or b, respectively, where b is the shorter span. Thus, the expression, M_b, refers to the moment on the short edge of the panel, or on a section parallel to the short edge, and not to the moment in the short span of length b.

All of the moment coefficients and distribution constants are tabulated and plotted as functions of the ratio of sides, b/a. Although in some instances values are given only for the range of b/a from 0.5 to 1.0, in most cases the necessary coefficients are given for all values of b/a.

Fixed-edge moments

The coefficients of average edge moments for panels fixed on all sides are given in Table A1 and plotted in Fig. A1. In both cases the values given are coefficients of wb^2, wherein w is the uniformly distributed load per unit of area, and b is the length of the short side. These moments are exact and were determined either directly or by interpolation from exact solutions obtained by means of the theory of elasticity.

Moments in simply-supported slab

The coefficients of average positive moments in simply-supported slabs are given in Table A1 and in Fig. A2. As for the fixed-edge moments, the values given are coefficients of wb^2. The moments considered are the maximum in each direction. In the short span, the maximum occurs on the section at midspan. In the long span, the maximum occurs at midspan for values of b/a greater than about 0.75; for smaller values the maximum moves toward the ends of the span, and for $b/a = 0.5$ or less it occurs at a distance of approximately 0.3 b from the short edge.

The positive moments given in Fig. A2 and Table A1 were determined from exact solutions obtained by means of the theory of elasticity.

Stiffness factors

The stiffness factors K_a and K_b represent the relation between the avearge moment and the average rotation on edges a and b, respectively. Specifically, $K = M_{av}/\Phi_{av}$, where M_{av} and Φ_{av} refer to the same edge of a panel, for which all other edges are fixed.

TABLE A1—NUMERICAL VALUES OF CONSTANTS FOR USE IN DISTRIBUTION PROCEDURE

Constant	b/a					
	0.5	0.6	0.7	0.8	0.9	1.0
Average fixed-edge moment:						
On long edge, M_a/wb^2	−0.0556	−0.0503	−0.0447	−0.0389	−0.0337	−0.0290
On short edge, M_b/wb^2	−0.0315	−0.0315	−0.0315	−0.0311	−0.0302	−0.0290
Average simple-span moment:						
In short span, M_a/wb^2	0.0651	0.0543	0.0444	0.0362	0.0293	0.0236
In long span, M_b/wb^2	0.0164	0.0175	0.0193	0.0213	0.0229	0.0236
Stiffness factors:						
On long edge, k_a	5.00	5.44	5.96	6.56	7.24	8.00
On short edge, k_b	7.50	7.60	7.70	7.80	7.90	8.00
Carry-over factors:						
Long edge to long edge, C_{aa}	−0.30	−0.24	−0.18	−0.12	−0.06	0
Long edge to short edge, C_{ab}	−0.30	−0.30	−0.30	−0.30	−0.30	−0.30
Short edge to short edge, C_{bb}	0	0	0	0	0	0
Short edge to long edge, C_{ba}	−0.15	−0.18	−0.21	−0.24	−0.27	−0.30
Positive moment correction factors:						
Moment on long edge, F_{aa}	0.250	0.210	0.170	0.130	0.090	0.050
F_{ab}	0.050	0.070	0.090	0.110	0.130	0.150
Moment on short edge, F_{bb}	0.025	0.030	0.035	0.040	0.045	0.050
F_{ba}	0.150	0.150	0.150	0.150	0.150	0.150

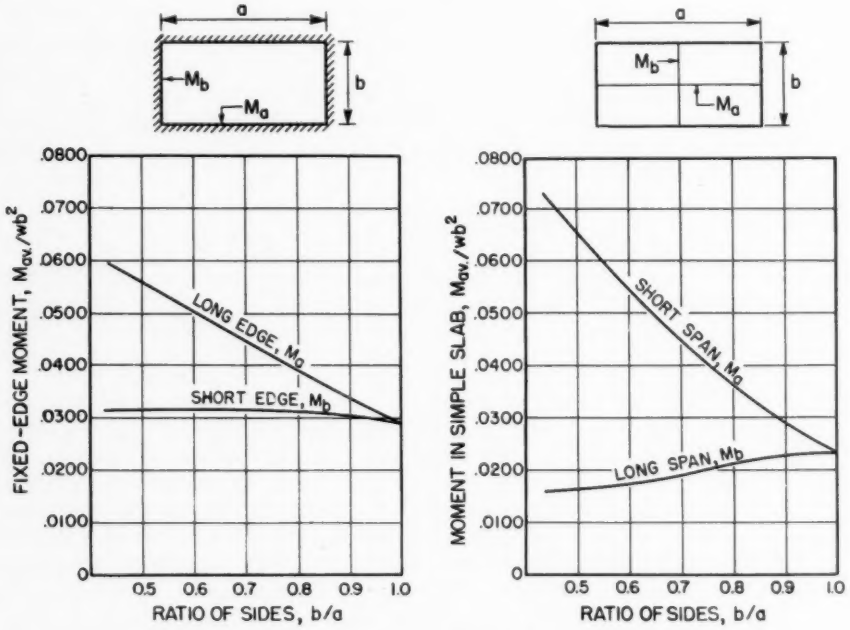


Fig. A1 (left)—Coefficients for fixed-edge moments

Fig. A2 (right)—Coefficients for moments in interior of simply-supported slab

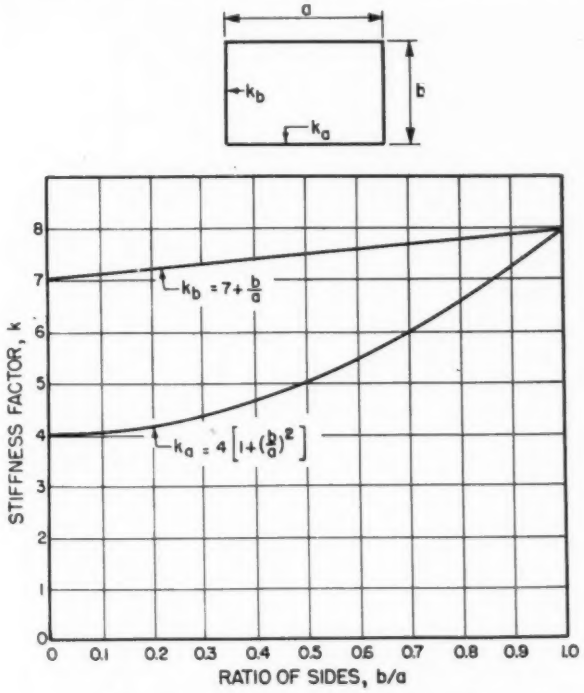


Fig. A3—Coefficients for stiffness factors

The stiffness factors for a given value of b/a are determined from the following expressions

$$K_a = k_a \frac{N}{b}$$

$$K_b = k_b \frac{N}{b}$$

in which $N = Et^3/12$, the stiffness of an element of the slab, and k_a and k_b are coefficients given in Table A1 and Fig. A3 as functions of b/a. Since the stiffness, K, depends on the shape of the panel as well as on the span length, the stiffnesses of continuous panels do not vary as much as for continuous beams.

Carry-over factors

Consider a rectangular panel having three edges fixed and the other simply supported. If a moment is applied to the simply supported edge, the moments induced on each of the fixed edges may be expressed as the product of the applied moment and the appropriate carry-over factor. Four such factors are required to define the behavior of the slab. For a moment applied on a short edge, b, the carry-over factors are C_{bb} for the opposite short edge, and C_{ba} for the adjacent long edges. Similarly, for a moment applied on a long edge, a, the factors are C_{aa} for the opposite long edge, and C_{ab} for the adjacent short edges. In each case, the first letter of the subscript refers to the edge on which the moment is applied, while the second letter refers to the edge to which the moment is carried over. The system of notation is further illustrated in Fig. A4.

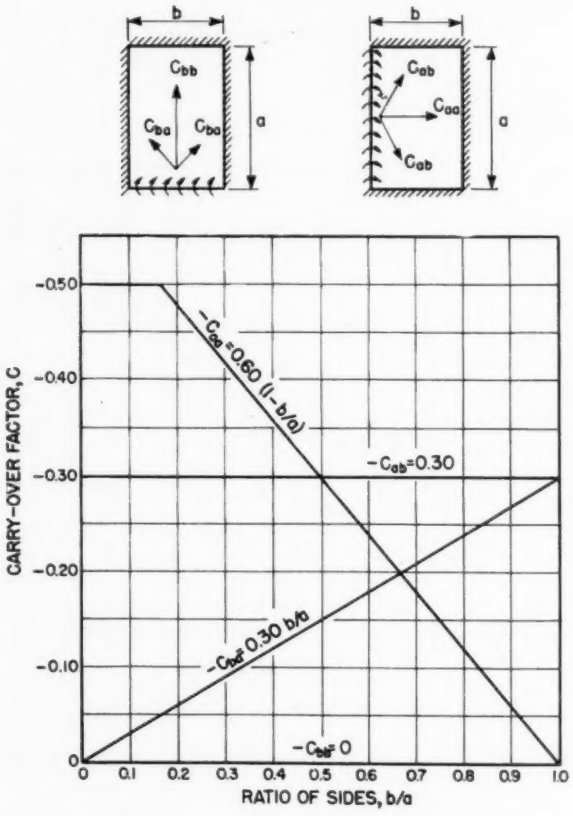


Fig. A4—Values of carry-over factors

Numerical values of the carry-over factors are given in Table A1 and Fig. A4. All of the values in question are negative in accordance with the sign convention of moment adopted for use with the distribution procedure: positive moment produces compression at the top of the slab.

Positive moment correction factors

The positive moment correction factors may be defined as the average moments produced at certain sections in the interior of a simply-supported rectangular panel by the application of a unit average moment to one edge of the panel. Numerical values are given in Table A1 and Fig. A5. The notation used is illustrated in the figure, and may be further explained as follows: If a moment is applied on a short edge, b, the positive moment correction factor applying to a section parallel to that edge is F_{bb}, and the factor for a section perpendicular to that edge is F_{ba}. Similarly, if the moment is applied on a long edge, a, the factors are designated F_{aa} and F_{ab} for the directions parallel and perpendicular to that edge, respectively.

Modified distribution constants

The values given in the preceding section for the stiffness and carry-over factors are for a panel with all edges fixed except the one to which the constants apply. If

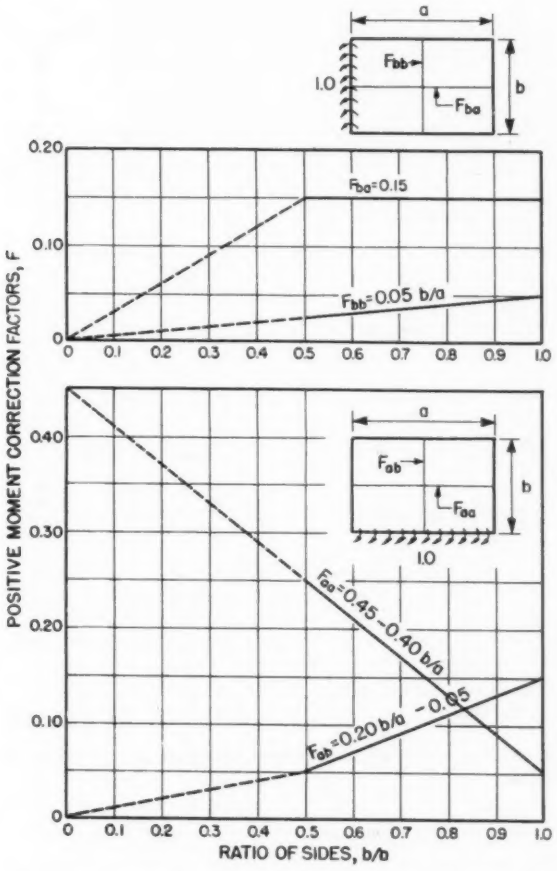


Fig. A5—Values of positive moment correction factors

one or more of these edges are simply supported, or if certain conditions of symmetry exist, modified values of the distribution constants may be used in order to simplify the numerical work involved.

Expressions have been derived for modified stiffness and carry-over factors for a number of common cases. The cases considered and the notation used are illustrated in Fig. A6. The algebraic expressions are given below for the constants on a short edge, b. Corresponding expressions applying to a long edge may be obtained simply by interchanging the subscripts a and b in each case.

Case 0. Basic panel:

Stiffness factor: k_b

Carry-over factors: C_{bb} and C_{ba}

Case 1. Opposite edge simply supported:

$$k'_b = k_b(1-C_{bb}^2)$$

$$C'_{ba} = \frac{C_{ba}}{1+C_{bb}}$$

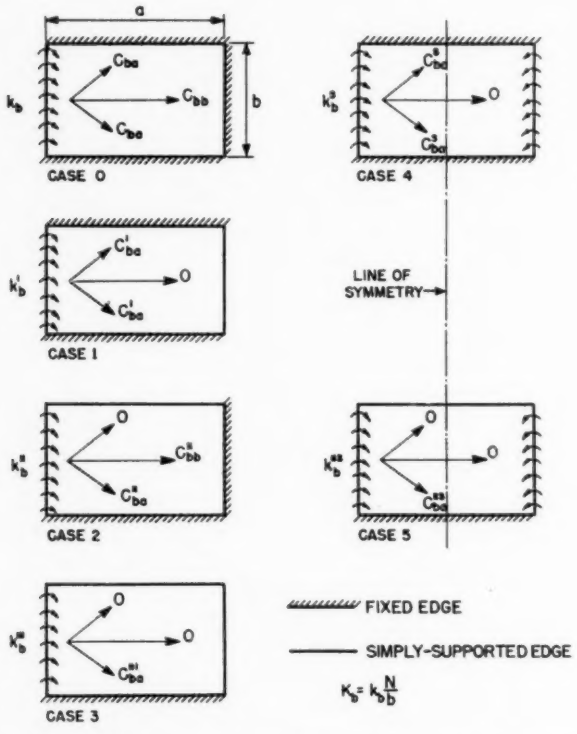


Fig. A6—Notation for modified stiffness and carry-over factors

Case 2. Adjacent edge simply supported:

$$k''_b = k_b\,(1 - C_{ab}C_{ba})$$

$$C''_{bb} = \frac{C_{bb} - C_{ab}C_{ba}}{1 - C_{ab}C_{ba}}$$

$$C''_{ba} = C_{ba} \cdot \frac{1 - C_{aa}}{1 - C_{ab}C_{ba}}$$

Case 3. Opposite and adjacent edge simply supported:

$$k_b''' = k''_b\,(1 - C''^2_{bb})$$

$$C_{ba}''' = \frac{C''_{ba}}{1 + C''_{bb}}$$

Case 4. Symmetrical conditions of loading and deformation:

$$k^s_b = k_b\,(1 + C_{bb})$$

$$C^s_{ba} = \frac{2\,C_{ba}}{1 + C_{bb}} = 2C'_{ba}$$

Case 5. Symmetry plus adjacent edge simply supported:

$$k_b''^s = k''_b\,(1 + C''_{bb})$$

$$C_{ba}''^s = \frac{2C''_{ba}}{1 + C''_{bb}} = 2C_{ba}'''$$

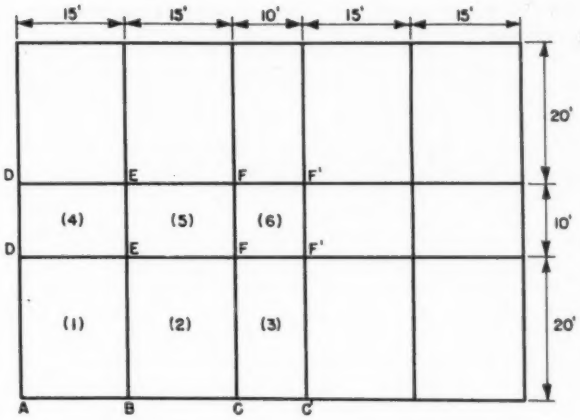


Fig. A7—Plan of slab for illustrative example

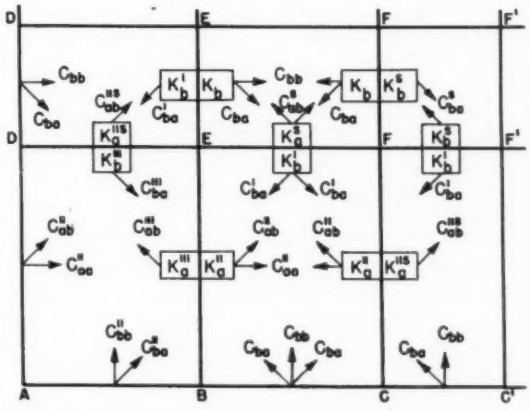


Fig. A8—Distribution constants for illustrative examples

The moments on the fixed edges of panels with one or more edges simply supported may be obtained from the values of fixed-edge moments by means of the distribution procedure. This calculation is easily performed as the first step in the solution.

Illustrative example

To illustrate its application, the distribution procedure will be used to compute moments in the continuous slab shown in Fig. A7. The loading considered will be $w = 100$ lb per sq ft, uniformly distributed over the entire area of the slab. The thickness of the slab is assumed to be the same for all panels.

The beams supporting the edges of the panels are assumed to be non-deflecting and to offer no restraint against rotation of the slab; that is, they have no torsional rigidity. The exterior edges of the slab are therefore simply-supported, and modified distribution constants may be used in the edge panels.

Since both the structure to be analyzed and the loading are symmetrical about both centerlines, only six of the fifteen panels need be considered. Furthermore, in panels

(3), (4), (5), and (6), modified stiffnesses and carry-over factors may be used to take account of the symmetrical conditions.

The particular distribution constants required in the solution of this problem are indicated by the appropriate symbols on the sketch in Fig. A8. Basic values of the carry-over factors were determined from the curves and formulas in Fig. A4, and values of the modified factors were computed by means of the equations in the preceding section. The resulting numerical values are given in Fig. A9(a).

The stiffness factors, k, were obtained from Fig. A3, and the quantity K was computed for a value of $N = 1$ since the slab thickness is equal for all panels. The numerical values of K thus obtained are also shown in the boxes in Fig. A9(a). The distribution factors which determine the proportion of the unbalanced moment to be distributed to each panel are computed as in the Cross method of moment distribution for continuous beams. For example: on edge BE the total stiffness is $0.369 + 0.390 = 0.759$, of which $0.369/0.759 = 0.49$ is contributed by panel (1); and $0.390/0.759 = 0.51$ by panel (2).

The calculation of the average fixed-edge moment was based on values of the coefficients of M_{av}/wb^2 obtained from Table A1 or Fig. A1.

All the necessary distribution constants are given on the sketch in Fig. A9(a), and the complete calculations for edge moments are given in Fig. A9(b). The operations carried out in these calculations may be described step by step as follows:

(a) Write in the average fixed-edge moment for each edge at a location adjacent to that edge as shown.

(b) Release the restraints on all exterior edges, which are simply supported in the actual structure, by balancing the moment on each edge to zero.

(c) Carry-over the proper proportion of the balancing moment from step (b) to the interior edges of each panel. In the corner panel (1), the carry-over factors are modified so that moments need not be carried over to the adjacent simply-supported edge.

From this point forward, the use of modified distribution factors for the interior edges obviated the carrying over of any moments to the simply-supported edges.

(d) Release the restraints on edge BE and distribute the unbalanced moment of -250 to the two panels in proportion to the distribution factors, thus: $+122$ or 49 percent to panel (1) and -128 or 51 percent to panel (2).

(e) Carry over the distributed moments to the remaining interior edges of panels (1) and (2). For example, in panel (2) carry-over $-128 \times -0.32 = +41$ to edge EF, and $-128 \times -0.23 = +29$ to edge CF.

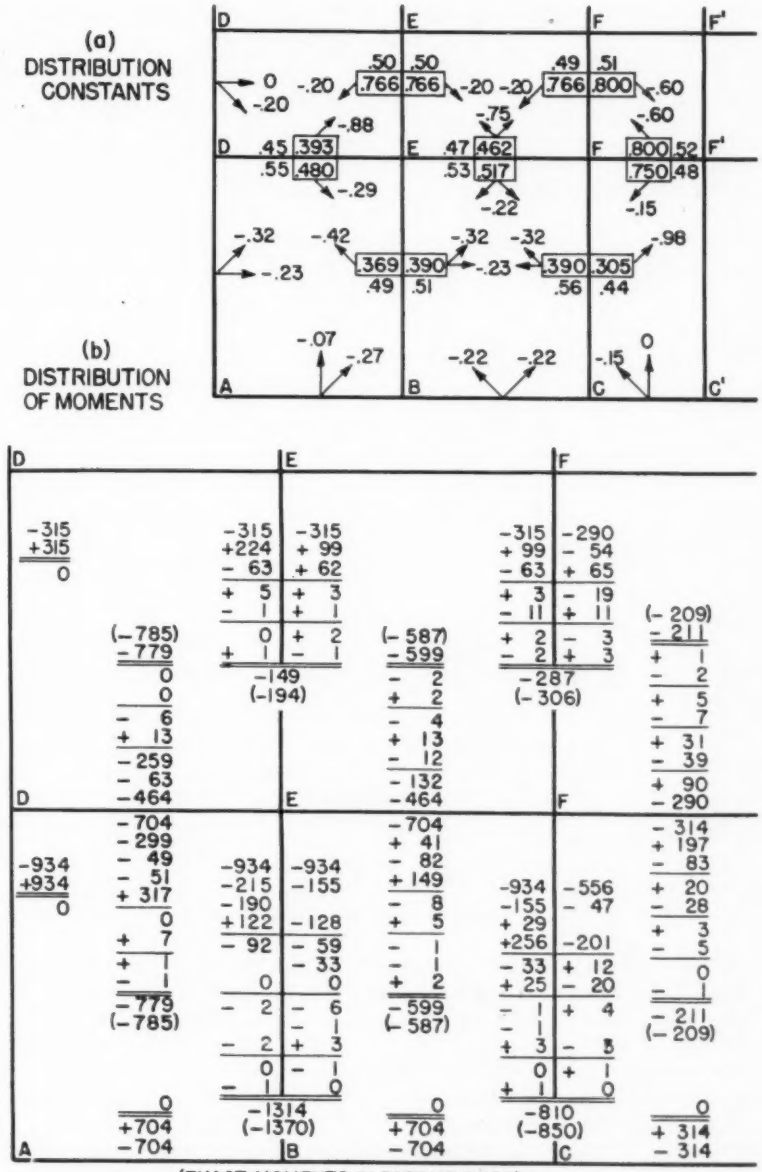
(f) Repeat the processes of steps (d) and (e) for edges CF, DE, EF, FF′, EE, and FF in turn, first balancing, then carrying over at each edge before proceeding to the next edge.

(g) Start over again at edge BE and repeat the process at successive edges until no unbalanced moment exists on any edge.

(h) Add all the moments in each panel at each edge. These are the desired average moments on the edges of the continuous slab.

In this problem, the procedure has been followed of distributing moments at each edge successively. In other problems it may be more convenient to release all edges simultaneously. The procedure in either case has a direct parallel with that used for continuous beams or frames.

The calculation of average positive moment on sections in the interior of each panel is illustrated in Fig. A10. The positive moment correction factors, F, obtained from Fig. A4, are given on the small sketches for each group of panels having the same



D		E		F	
-315		-315	-315	-315	-290
+315		+224	+ 99	+ 99	- 54
0		- 63	+ 62	- 63	+ 65
		+ 5	+ 3	+ 3	- 19
		- 1	+ 1	- 11	+ 11
	(-785)	0	+ 2	+ 2	- 3
	- 779	+ 1	- 1	- 2	+ 3
	0	-149		-287	
	0	(-194)		(- 306)	
	- 6		- 4		
	+ 13		+ 13		
	- 259		- 12		
	- 63		- 132		
D	-464	E	-464	F	
	- 704		- 704		
	- 299		+ 41		
-934	- 49	-934	- 82		
+934	- 51	-215	+ 149	-934	- 556
0	+ 317	- 190	- 8	-155	- 47
	0	+122	+ 5	+ 29	
	+ 7	- 92	- 1	+256	-201
	+ 1	- 59	- 1	- 33	+ 12
	- 1	- 33	+ 2	+ 25	- 20
	- 779	0	- 599	- 1	+ 4
	(-785)	- 2	(- 587)	- 1	
		- 6		+ 3	- 8
		- 2		0	+ 1
		+ 3		+ 1	0
	0	0	0		
		- 1		-810	
	+704	-1314	+704	(-850)	
A	-704	B	- 704	C	

- 314	
+ 197	
- 83	
+ 20	
- 28	
+ 3	
- 5	
0	
- 1	
- 211	
(- 209)	
0	
+ 314	
- 314	

(EXACT MOMENTS IN PARENTHESES)

Fig. A9—Illustrative computation sheet

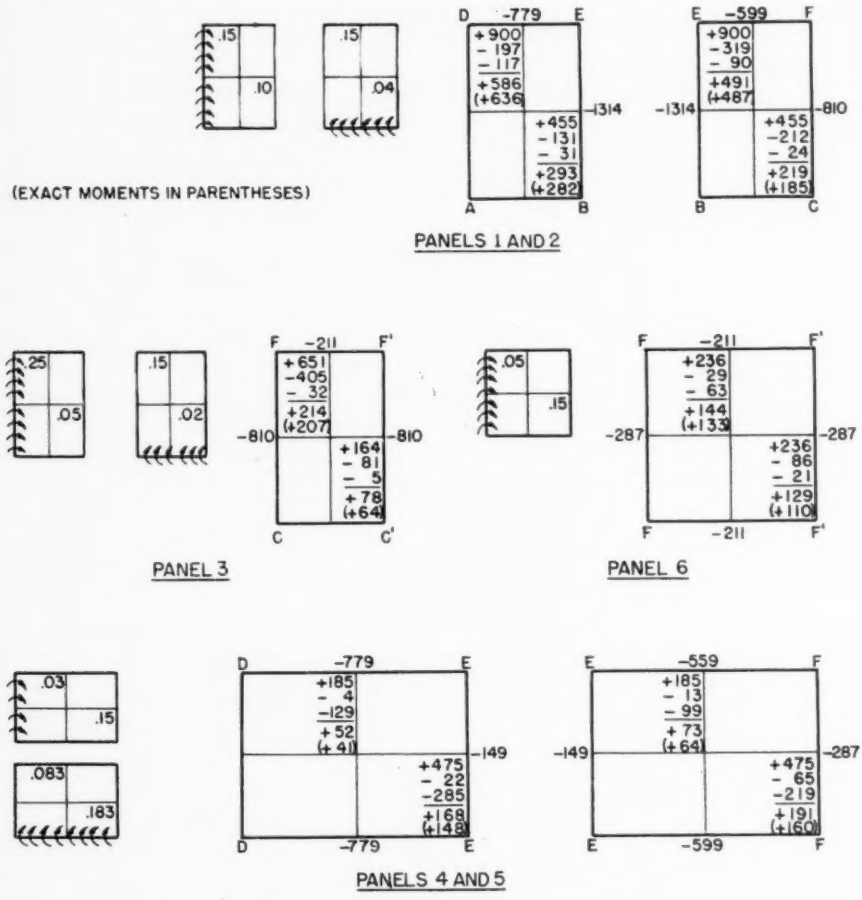


Fig. A10—Calculation of positive moments

values of b/a. Average positive moments, computed from the coefficients in Table A1 or Fig. A2, are written on a sketch of each panel adjacent to the section to which they refer. Corrections due to the edge moments, obtained as the product of those moments and the appropriate factor, F, are written separately for the moments on the long and short edges. The net average positive moment in each direction is obtained as the sum of the positive moment for the simply-supported slab and the corrections due to the edge conditions.

Correct values of the average moments are given in parentheses immediately below the moments obtained from the distribution procedure in Figs. A9(b) and A10. These correct moments were taken from an exact solution obtained by C. W. Pan,* and the writers are indebted to Professor L. C. Maugh of the University of Michigan for permitting their use herein.

The ratio of approximate to exact average edge moment in Fig. A9(b) ranges from 0.94 to 1.02, except for the one low value of 0.77. Although this one error is rather

*Pan, C. W., "Analyses of Continuous Slabs," A dissertation submitted in partial fulfillment of the requirements for the degree of Doctor of Science in the University of Michigan, Ann Arbor, 1939.

large, it should be noted that the moment in question is relatively small and that the numerical value of the difference in moment is not excessive when compared with the other cases. The ratio of approximate to exact average positive moment in Fig. A10 ranges from 0.92 to 1.19 in the short span and from 1.04 to 1.27 in the long span. In general, the greatest errors are associated with relatively small moments. The fact that the approximate moments are nearly always larger than the correct values is the result of a deliberate attempt to be conservative in this phase of the procedure, partly because of the greater importance of the positive moments, and partly to allow for some uncertainty regarding the location of the section for maximum moment in slabs with various edge conditions. The excess of approximate over exact positive moments in this example is also due in some measure to the fact that the approximate negative moments are generally smaller than the exact values.

Discussion of a paper by C. P. Siess and N. M. Newmark:

Rational Analysis and Design of Two-Way Concrete Slabs*

By MOUNIR BADIR, PAUL W. NORTON and AUTHORS

By MOUNIR BADIR†

The purpose of this discussion is to examine the data given by the authors as a measure of the increase in the moments in an interior square panel of a continuous reinforced concrete slab, as a result of the deflection of the supporting beams. As given in Table 2 of the paper, the effect of beam deflection is to increase the average positive and negative moments by 21 and 12 percent respectively.

The discussion presented herein is based on a theoretical investigation on the effect of the deflection of the beams on the moments in homogeneous elastic slabs. For this investigation 16 typical interior square panels were designed in accordance with the ACI Building Code, 1946. In each panel the slab rested on four beams which were supported on columns at the corners of the panel. The spans and the live loads were so chosen as to cover a wide range of the actual conditions present in practice. The spans were 15, 20, 25 and 30 ft. For each case the loads had intensities of 50, 100, 200 and 400 lb per sq ft. In the analysis of six of the slabs, the moments of inertia of the beams were computed according to two assumptions. In the first, the cross section was considered a T-section, and in the second it was assumed a rectangular section. For the remaining ten slabs only a rectangular section was considered. Thus a total of 22 panels was available for the analysis. The purpose of these designs was to obtain reasonable values of relative stiffness of beam and slab for the further calculations.

Each panel was analyzed and the moments at several points on the slab and the beam were determined. The designs and the calculations were made by the graduate students at the University of Illinois in a course taught by Prof. N. M. Newmark.

*ACI Journal, Dec. 1948, *Proc.* V. 45, p. 272. Disc. 45-15 is a part of copyrighted Journal of the American Concrete Institute, V. 21, No. 4, Dec 1949, Part 2, *Proceedings* V. 45.

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The moments in a typical interior square panel with infinitely rigid supporting beams were determined at the same points as in the other structures for comparison of the moment coefficients. The difference between the moments in this case and the moments in the above mentioned 22 panels is the effect of the deflection of the beams on the values of the various moments.

The investigation was made by use of the theory of bending of plates, after transforming the equations into the usual finite difference form. The load considered was a uniformly distributed one covering the whole slab. The slab was subdivided into 8 x 8 squares. The choice of this number of divisions was arbitrary; however, it was believed to yield results with a good degree of accuracy, and to involve only a reasonable amount of calculation. The computations were made for a value of zero for Poisson's ratio.

The results of the analysis were used to plot the moments at various points on the slab and the beam against H, which is the ratio of the flexural rigidity of the beam to that of a width of the slab equal to the side of the panel.

The coefficients for the average positive moments along the middle line of the slab and the average negative moments along the edge are given in Table A for values of H equal to infinity, 4, 3, and 2. The average moment coefficients were computed by Simpson's rule. The last three columns give the percentage increases in the moments as a result of the deflection of the beams for the case of H equal to 4, 3 and 2. Column 5 gives the percentage increase in the same moments as given by the authors in Table 2 of the paper. Comparison with the corresponding figures of columns 6, 7 and 8 shows that the percentage increase in the average moments as given by the authors is in fairly close

TABLE A—COMPARISON OF AVERAGE MOMENTS FOR SEVERAL VALUES OF BEAM STIFFNESS

	Coefficient for average moment M_{av}/wb^2				Increase in moment due to beam deflection, percent			
	$H = \infty$	$H = 4$	$H = 3$	$H = 2$	Authors	$H = 4$	$H = 3$	$H = 2$
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Positive moments at midspan	0.0103	0.0132	0.0156	0.0166	21	28	51	61
Negative moments at edge	0.0283	0.0321	0.0335	0.0365	12	13.5	18	29

agreement with the results of the theoretical investigation for the case in which H is equal to 4. If H is greater than 4 the percentages given by the authors are on the safe side.

In all cases the values of H were greater than 4 where the beam stiffness was computed for a T-section. Only in several extreme cases for rectangular beam sections was the value of H less than 4.

By PAUL W. NORTON*

The comment I have to make is this: I do wish that those who are presenting results in the analysis of two-way slabs would express the moment in a strip of the slab in terms of the span of that strip. It would seem to any of us strange to have to express the moment in a banking house girder, for example, in terms of the span of the Brooklyn Bridge. It seems to me analogously unilluminating to have to express the moment in a strip the long way of the rectangular slab in terms of the span of the short-way strip. If the moment in the strip were expressed in terms of its own length, it would then be appropriate to express the ratio of the sides of the panel always as the ratio of its transverse dimension to the span of the strip under consideration.

The advantage of doing this seems to me so obvious as to defy refutation. Each strip of the slab is a beam; the moments to which it is subject depend on its span, on the amount and distribution of the load it carries and on its conditions of end restraint. But of these three, the last named are affected by several variables; the transverse widths of the panels, relative to the span of the strip considered, and also conditions of restraint at the sides of the panel affect the share of the whole load ascribed to the strip itself. The length, loading, and restraints of adjacent spans, and the flexibility of supports, affect both the loading and the end restraint of the strip. It is the object of the research summarized in this paper to discover how these variables affect the moments, shears and requirements of design in the strip. The one thing that is invariable is the span; why then choose the span of anything else as that in terms of which to express the moment?

In studying the effect of the ratio of transverse dimension to span, this ratio may be varied continuously from a useful minimum of say one-third or one-half to an upper limit of two or three. And if it be objected that it is desirable to investigate a variation outside these limits, and that the scale of graphical presentation of results is inconvenient unless the objectionable device is adopted of turning the ratio upside down, let me make this suggestion: if the results are shown graphically with the ratio plotted to logarithmic scale, the objection is overcome, and other advantages are gained. Unity is in the middle of

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the chart and the ratio one third is as far to the left as three is to the right. A little study has convinced me that this presentation, and particularly the continuity of the presentation of results for long-way strips and short, with the dimension ratio passing from small values through unity to large values results in increased clarification of its effects.

AUTHORS' CLOSURE

The data presented by Dr. Badir on the effects of beam deflection are greatly appreciated by the writers. The procedure used in the paper for the calculation of these effects was admittedly approximate and, in some respects, rather crude. It is therefore very gratifying to find that the values given in Table 2 for a square panel agree so closely with the more accurate values given in Table A of the discussion, for a value of $H = 4$. The use of this value of H as a basis for comparison is justified by Dr. Badir's statement that the values of H for typical designs were nearly always greater than 4. A similar conclusion was reached by the writers from their studies of other designs, and a value of $H = 4$ was used as a minimum figure in computing the effects of beam deflection by the procedure described in the paper.

The more exact values presented by Dr. Badir refer only to a square panel, and consequently afford a direct check on only two of the six values given in Table 2. However, the calculations for the effect of beam deflection were based on a consistent set of assumptions and the procedure used was rational in form; therefore it is not unreasonable to believe that the remaining values in Table 2 are reliable.

Mr. Norton feels that the moment in the direction of the long span of a slab should be expressed in terms of the length of that span rather than in terms of the length of the short span. The latter procedure has been used by the writers and by other investigators of moments in plates. There are several arguments that might be presented in favor of this general practice, but the best and most obvious is that the moments in the long direction of a *slab* simply do not depend in any direct manner on the length of the long span. They do depend on the length of the short span, however.

As Mr. Norton points out, the moments in a *beam* depend, among other things, on (1) the amount and distribution of the load carried by the *beam*, and (2) the span length of the *beam*. However, if the beam in question is not a real beam but only an imaginary strip extending in the long direction of a two-way slab, the amount and distribution of the load carried by the strip depend also on the span length. It so happens, in this case, that the relations between load, span, and moment are such that the moment in the long direction *in a slab* is practically independent

of the span length in that direction. On the other hand, this moment is strongly dependent on the span length in the short direction. Consequently, it would be just as illogical to relate this moment directly to the long span when dealing with a *slab*, as it would be to ignore this relation when dealing with a *beam*.

Mr. Norton also raises an objection to the manner of expressing the dependence of slab moments on the ratio of sides, b/a. The writers have followed Prof. Westergaard's lead in this respect. Their approach emphasizes the facts that the behavior of a slab in the long and short spans is fundamentally different, and that the moments in a rectangular panel are related in a definite manner to the shape of the panel. The expression of moments in a *slab* as moments in a *beam strip* of some *width and length* is artificial, and serves only to conceal the true relations. Actually, moments in a slab in either direction are most directly functions of the short span and the ratio of sides, and a clear picture of the behavior of a rectangular slab can be obtained easily by considering these relations.

It is important to remember that the structure in question is a *slab*, and that a *slab* behaves in a manner different from a *beam*. The writers must admit, however, that it has not always been easy for the designer to make this distinction, which has frequently been obscured by methods of design and analysis which consider a slab to be simply an assemblage of beams. The writers were particularly careful in this paper to avoid the beam-strip concept of slab action, either in analysis or design. Both the proposed design procedure and the distribution procedure used for the analysis deal with average moments on the entire width of the slab, rather than with the moments in imaginary beam strips.

The writers believe that better understanding and more effective use of two-way slab construction will result from better understanding of the behavior of slabs. It will be found after very little study, that the true action of a slab, considered as a single element, is actually much simpler to understand and work with than is the confusing and sometimes misleading concept of a slab as a complicated assembly of beams.

What are the corrosion effects of phenol, molasses, lard, milk and other reagents? Based on tests of sample floor specimens the authors make recommendations for use of concrete in floors subject to chemical attack.

Corrosion Resistance Tests of Concrete Floors—With and Without Metallic Aggregate*

By E. W. SCRIPTURE, Jr.,† and C. H. SAKRYD‡

SYNOPSIS

An attempt has been made to determine the conditions under which concrete floors can be expected to give good service. Resistance to abrasion was determined on specimens with and without metallic aggregate after exposure to typical reagents selected from the groups described by the Joint Committee as having corrosive effects of an intermediate nature between no attack and severe attack. On the basis of the experimental results the reagents in the Joint Committee report have been classifled in relation to anticipated service of concrete floors.

INTRODUCTION

The effects on concrete of many of the reagents commonly encountered in industry are the subject of an appendix to the 1940 report of the Joint Committee.§ The observations on these reagents are qualitative, not quantitative, and it is possible that some significant effects would be absent on really dense concrete floors. For reagents in the list described as not attacking concrete on the one hand and those described as rapidly disintegrating concrete on the other a prediction of the serviceability of a concrete floor is relatively simple. For a large number of reagents, however, the Joint Committee uses such expressions as "attacks slowly", "slight attack", and so forth, from which it is difficult to know whether a concrete floor would stand up under the specific reagent. A typical reagent from each group listed by the Committee, except those which were described as not attacking concrete at all, and most of those described as attacking concrete severely and rapidly, was selected for test of resistance of floor specimens to attack.

*Received by the Institute June 2, 1948. Title No. 45-16 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 4, Dec. 1948, *Proceedings* V. 45. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than April 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

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§Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, American Concrete Institute 1940.

TEST PROCEDURE

Slabs were made up in rectangular baking tins 8x8x1½ in. from a 1:1½:1½ cement-sand-pea gravel mortar compacted and floated in a normal manner. One-third of the slabs was left plain, one-third was finished with 60 lb per 100 sq ft of a metallic aggregate combined with a cement dispersing agent, and the remaining third with 120 lb per 100 sq ft of the metallic aggregate applied as dust coats consisting of two parts metallic aggregate to one part cement by weight. Without the cement dispersing agent it is not possible to incorporate the heavier shake into a dry mix. The slabs were troweled, cured 10 days, allowed to dry out, and then a dike of Permatex to hold the reagent was put around the edges. These slabs, three for each condition, were exposed to the various selected reagents for six months. They were observed daily and additional reagent added where necessary to keep the slab covered. Weekly the slabs were washed to remove any deposits which had formed and fresh reagent added. For those reagents which showed considerable corrosion by visual inspection in a shorter period than six months, the test was discontinued when a substantial amount of corrosion appeared to have occurred. At the conclusion of the six months corrosion period all slabs were washed, observed visually for surface condition, and measured to determine depth of corrisoon. Abrasion tests were then made by the method regularly used in this laboratory, which has previously been described.*

DISCUSSION

Visual observation of the surface condition of the slabs after corrosion was not very enlightening. In most cases, for the reagents listed in Table 1, little or no effect on the surface was observable. In a few cases small amounts of pitting or blistering or both were observable but these could not be correlated with the results of the wear tests, except that the metallic aggregate slabs exposed to magnesium chloride, which showed poor abrasion resistance, were badly blistered. In a few cases the metallic aggregate specimens showed slight rusting but this did not affect wear resistance. The metallic aggregate specimens showed visual evidence of corrosion in more cases than the plain specimens but this is attributable to the greater contrasts developed where metallic aggregate was used rather than to any real differences in amount or type of corrosion. For the strongly corrosive reagents of Table 2, bad pitting was observed in all cases.

The measurements of the depth of corrosion were even less informative. In all cases, for reagents of Table 1, the measured differences, which

*Scripture, Edward W. Jr., "Metallic Aggregate in Concrete Floors," ACI JOURNAL, Sept.-Oct. 1936, *Proc.* V. 33, p. 17.

TABLE 1—CORROSION RESISTANCE OF CONCRETE FLOORS—6 MONTHS EXPOSURE

Reagent	Depth of abrasion after corrosion, in.		
	Metallic aggregate		
	None	60 lb per 100 sq ft	120 lb per 100 sq ft
None	.009	.004	.003
Phenol (10 percent)	.009	.003	.003
Molasses (10 percent)	.007	.003	.002
Sugar (10 percent)	.007	.003	.003
Carbon dioxide (wet)	.009	.003	.003
Sodium sulfate (10 percent)	.008	.004	.004
Lard oil (100 percent)	.009	.008	.005
Magnesium sulfate (10 percent)	.008	.006	.005
Magnesium chloride (10 percent)	.011	.012	.008
Milk (100 percent)	.010	.005	.005
Waste sulfite liquor*	.020	.004	.003
Sulfide ore (wet)	.020	.004	.004
Cottonseed oil (100 percent)	.020	.032	.028

*Concentration as produced—approximately 10 percent solids.

TABLE 2—CORROSION RESISTANCE OF CONCRETE FLOORS—REAGENTS HAVING RAPID CORROSION EFFECTS

Reagent	Time of exposure, days	Depth of abrasion after corrosion, in.		
		Metallic aggregate		
		None	60 lb per 100 sq ft	120 lb per 100 sq ft
None	—	.009	.004	.003
Ammonium sulfate (10 percent)	89	.020	.010	.015
Sulfuric acid (10 percent)	47	.017	.006	.004
Hydrochloric acid (10 percent)	47	.017	.009	.009

were both plus and minus, were within the limits of accuracy of the measurements except in the one instance of one set of metallic aggregate specimens (60 lb per 100 sq ft) exposed to magnesium chloride, which showed significant corrosion. Even with the strong reagents of Table 2 the measured depth of corrosion was only of significant magnitude with sulfuric acid, even though severe pitting was visually apparent for all three reagents.

In view of the lack of significance of the visual observations and measurements of the depth of corrosion, reliance was placed on the degree of resistance to abrasion after corrosion. The results of the corrosion tests with all agents applied for the full six-month period are given in Table 1.

TABLE 3—CORROSION RESISTANCE OF METALLIC AGGREGATE FLOORS—6 MONTHS EXPOSURE

Type of exposure	Depth of abrasion after corrosion, in.	
	Metallic aggregate	
	None	60 lb per 100 sq ft
None	.010	.004
Picric acid (sat. sol'n)	.010	.002
Ammonium nitrate (sat. sol'n)	.030	.007
None	.011	.002
Cutting oil (sulfurized)	.015	.005
Lactic acid (10 percent)	.021	.013
None	.010	.002
Milk (100 percent)	.011	.004
Acetic acid (10 percent)	.062	.017

TABLE 4—CONCRETE FLOORS TO BE RECOMMENDED—REAGENTS* HAVING NO EFFECT ON CONCRETE

Typical reagent tested	Reagents in group (Joint Committee classification)
None	Petroleum oils Light fuel oils Volatile oils—kerosene, benzene, naphtha, gasoline Heavy oils
None	Coal-tar distillates Benzol, toluol, xylol, cumol Pitch, anthracene, carbonzol, paraffin
None	Organic materials Oxalic acid Carbonic acid (dry)
None	Salts Chlorides of sodium, potassium, strontium, calcium Nitrates of calcium, potassium, sodium Carbonates Fluorides Silicates Sulfides except ammonium
None	Miscellaneous Ammonia water Wood pulp Tanning liquors (non-acid) Alcohol

*As described by Joint Committee

TABLE 5—CONCRETE FLOORS TO BE RECOMMENDED—REAGENTS* SHOWING SLIGHT OR MODERATE ATTACK ON CONCRETE

Typical reagent tested	Reagents in group (Joint Committee classification)
Phenol	Coal-tar distillates Phenol, cresol, lysol, creosote Carbolineum
Molasses Sugar	Miscellaneous Molasses, sugar, corn syrup, glucose
Sulfite liquor†	Miscellaneous Sulfite liquor
Milk	Miscellaneous Milk
Sodium sulfate Magnesium sulfate	Salts Sulfates of calcium, potassium, sodium, magnesium, copper, zinc
Sulfide ore (wet)†	Salts Sulfide ore
Ammonium nitrate†	Salts Nitrate of ammonia
Lactic acid (dilute)† Acetic acid (dilute)†	Organic materials
Picric acid Cutting oils	Not classified by Joint Committee
Linseed oil Resin	Organic materials No test made as published data available to show resistance of concrete to these reagents

*As described by Joint Committee
†Metallic aggregate floors only

TABLE 6—CONCRETE FLOORS NOT RECOMMENDED—REAGENTS* SHOWING SEVERE ATTACK ON CONCRETE

Typical reagent tested	Reagents in group (Joint Committee classification)
Sulfuric acid Hydrochloric acid	Inorganic acids Sulfuric, nitric, sulfurous, hydrochloric, hydrofluoric
No reagent tested	Salts Acid sulfates (and other strongly acid salts)
Ammonium sulfate	Salts Sulfates of ammonia
Magnesium chloride	Salts Chlorides of magnesium, iron, mercury, copper, ammonia
Cottonseed oil	Organic materials Vegetable oils, cottonseed, poppyseed, walnut, soy bean, peanut, rapeseed, olive, cocoanut, almond

*As described by Joint Committee

These reagents fall into three groups. The first, which includes phenol, molasses, sugar, lard oil, carbon dioxide (wet), and sodium sulfate (10 percent), shows little or no effect on the floor specimens. The second, which includes magnesium sulfate (10 percent), magnesium chloride (10 percent), and milk, has comparatively moderate corrosive effects. It is somewhat surprising to note that the effects of magnesium chloride are slightly more serious than those of magnesium sulfate and also that the abrasion resistance of the lighter metallic aggregate shake is about the same as that of the plain slab after corrosion with this reagent. The third group, waste sulfite liquor, sulfide ore (wet), and cottonseed oil, has serious corrosive effects. In two cases, waste sulfite liquor and sulfide ore, the metallic aggregate surfaces, however, are not appreciably affected. On the other hand, cottonseed oil appears to have a more severely corrosive effect on metallic aggregate surfaces than on the plain specimens. The observed results with cottonseed oil are in agreement with previous observations as given by Lea and Desch* but this is not the case for lard oil.

While it is well known that strong acids attack concrete, two examples and ammonium sulfate were included in the test program for comparison. The results with these are shown in Table 2.

It will be seen that the two strong acids, as expected, and ammonium sulfate attack the concrete very rapidly. That the metallic aggregate surfaces show less attack than the plain surfaces, especially with the acids, is not significant in view of the short duration of the corrosion period, and that the visual inspection revealed serious pitting. The floors would go to pieces with excessive rapidity in any event, with or without metallic aggregate.

Some additional data, not included in the single series described above, but obtained in separate experiments made previously, are given in Table 3.

In these tests the good resistance to unfermented milk found in the other series is confirmed, as it is also by experience in food industries, if the floors are kept clean. Good resistance to picric acid, to cutting oil, and for the metallic aggregate floor to ammonium nitrate is found. Fair resistance to weak organic acids such as lactic and acetic acids is shown by the metallic aggregate specimens.

CONCLUSIONS

This work seems to provide sufficient quantitative data to permit predictions regarding the probable satisfactory or unsatisfactory service which a concrete floor will give under most corrosive conditions com-

*Lea, F. M., and Desch, C. H., *The Chemistry of Cement and Concrete*, p. 386 (Edward Arnold & Co., London).

monly encountered in industry. On this basis Tables 4 and 5 have been prepared showing the reagents for which concrete floors would be recommended, and Table 6 showing reagents for which a concrete floor of any kind would not be recommended. These tables are based on the assumptions that:

1. If the Joint Committee describes the reagent as having no disintegrating effect on concrete, a concrete floor will give satisfactory service.

2. If the test data show that in six months the reagent has no adverse effect, or only a small adverse effect, then a good concrete floor will give satisfactory service.

3. If the reagent tested shows a substantial effect in decreasing abrasion resistance within six months or less, then a concrete floor will not give satisfactory service.

4. The effects of all the reagents in a related chemical group, as classified by the Joint Committee, will be, within reasonable limits, similar and approximately the same as that of the typical reagent used in the test program.

In view of the rather unexpected deterioration observed with magnesium chloride, the group of chemicals represented by this compound was put in the list where concrete floors would not be recommended, pending further data. Similarly, the vegetable oils are put in the same category in view of the severe effects of cottonseed oil. It seems probable that some of the other vegetable oils would not seriously attack concrete floors, but without further data it would seem inadvisable to recommend concrete floors under such exposure.

As a general statement, concrete floors can be recommended where:

1. The rate of corrosion is known to be low.

2. Specific data, such as those given as the result of this work, show that a concrete floor will give good service.

3. The corrosive action is dependent on prior fermentation of an organic material and the floor will be kept clean.

4. The corrosive condition is moderate and the resistance to mechanical abrasion is of primary importance.

A concrete floor should definitely not be recommended where it will be exposed to corrosive conditions on which no quantitative data are available relating directly to floors.

Mr. Price sets forth a development in masonry units which may be merged to form an essentially monolithic member. Suitable for floor, roof or wall, these slabs may carry radiant heating units in their hollow cores.

Flexicore® Floor and Roof Slabs*

By GAYLE B. PRICE†

SYNOPSIS

This paper describes briefly the design, manufacture, erection procedures and use of Flexicore slabs in precast concrete floor and roof systems. The Flexicore slab has a standard cross section 12 in. wide and 6 in. thick, lightened by two holes 4⅝ in. in diameter running the full length of the slab which is made in inch variations of length up to a maximum of 22 ft 6 in. Reinforcement consists of two bars in the top of the slab for "handling" stresses and two or three prestressed tension bars as required at the bottom to take care of bending moment.

INTRODUCTION

When development work was started in 1938 on Flexicore slabs four major requisites were considered as essential for a successful precast floor and roof system: (1) a unit that could fit into a wide variety of uses, (2) a system that would require a minimum of field work and could be easily and rapidly erected, (3) a system which would be easy to detail so that layout work would be minimized and (4) a unit that could be used as a finished ceiling.

Although the development work was curtailed during the war the recent demands for this comparatively new product indicate the accuracy of this evaluation. As of December, 1947 approximately 2,500 separate jobs ranging from 200 sq ft to 500,000 sq ft and totaling around 8,000,000 sq ft have been erected. These jobs cover a wide variety of applications as the adaptability of this unit has exceeded the original expectations.

DESCRIPTION

Flexicore slabs have a standard cross section 12 in. wide and 6 in. thick. (Fig. 1.) Two holes 4⅝ in. in diameter run the full length of the slabs which are made in inch variations of length up to a maximum

*Presented at the ACI 44th Annual Convention, Denver, Colo., Feb. 23, 1948. Revised manuscript received Sept. 7, 1948. Title No. 45-17 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 4, Dec. 1948, *Proceedings* V. 45. Separate prints are available at 35 cents each **Discussion** (copies in **triplicate**) should reach the Institute not later than April 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

†Member American Concrete Institute, President, The Flexicore Co., Inc., Dayton, Ohio.

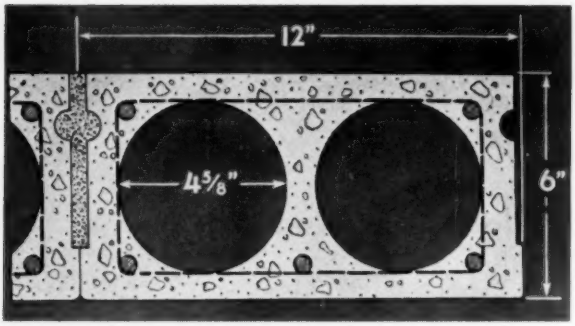

Fig. 1—Cross section of Flexicore slab showing grouted key joint

of 22 ft 6 in. Reinforcing steel consists of two bars in the top of the slab for "handling" stresses and two or three prestressed tension bars at the bottom to take care of bending moment. (Fig. 2.) Sizes of the bars are varied according to live load and span requirements but the concrete cross section remains constant. For average loads, stirrups are not required but they are added for heavier load conditions.

DESIGN DATA

The basis of design for Flexicore slabs was taken from "Building Regulations for Reinforced Concrete (ACI 318-41)" as follows:

Concrete:	Compression strength—28 days	= 3750 psi
	Extreme fiber working stress	= 1500 psi
	Shear stress (bars anchored by having lugs welded to ends)—3 percent of 3750	= 112.5 psi
Steel:	Yield point	= 50,000 psi
	Tensile working stress	= 20,000 psi
	Compression working stress	= 8 × concrete stress
	Ratio E_s/E_c	= 8

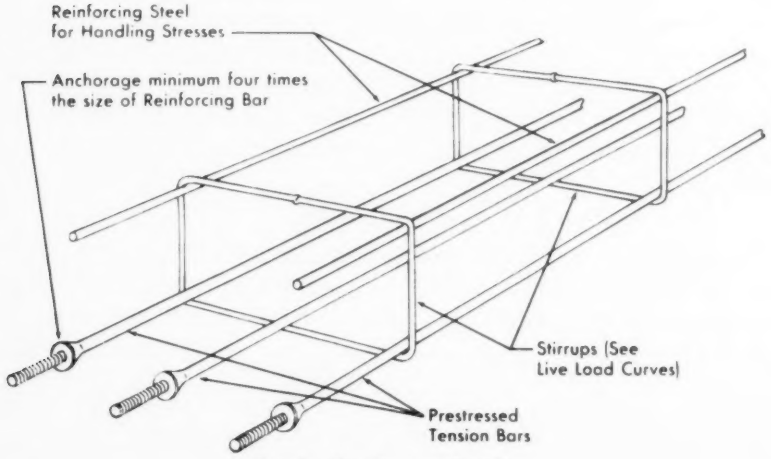

Fig. 2—Reinforcing details

TABLE 1—ELEMENTS OF DESIGN—6 IN. x 12 IN. SECTIONS

Depth to Steel = 5⅛ in. all sections. Net concrete thickness at top = ¾ in.
$f_s = 20{,}000$; $f_c' = 3750$; $E_s/E_c = 8$; $E_s = 30{,}000{,}000$.
$\Delta = \frac{22.5\ WL^4}{EI}$ W in lb per lineal ft; L in ft; Δ in inches.

Unit mark	Tension steel, sq in.	Compr. steel, sq in.	d	d'	kd	jd	Compr. area, sq in.	Centroid above axis, in.	Ave. compr. stress, psi	f_c, psi	Safe moment, in. lb	I_s, in.⁴	I_c, in.⁴
F 15	3 ¼ φ = 0.147	2 ¼ φ = 0.098	5.125	0.875	0.92	4.775	10.99	0.570	267	507	14,050	2.95	23.76
F 23	3 5/16 φ = 0.230	2 ¼ φ = 0.098	5.125	0.875	1.11	4.760	12.39	0.745	372	720	21,900	4.38	34.89
F 33	3 ⅜ φ = 0.331	2 ¼ φ = 0.098	5.125	0.875	1.299	4.709	13.59	0.883	488	908	31,150	5.96	47.21
F 42	2 ⅜ φ 1 ½ φ = 0.417	2 5/16 φ = 0.153	5.125	0.906	1.422	4.673	14.70	0.970	568	1050	39,000	7.20	56.92
*F 47	2 ⅜ φ 1 ½ □ = 0.471	2 5/16 φ = 0.153	5.125	0.906	1.492	4.658	15.05	1.025	627	1140	43,900	7.95	62.65
F 53	2 ⅜ φ 1 ⅝ φ = 0.528	2 5/16 φ = 0.153	5.125	0.906	1.566	4.642	15.36	1.083	688	1245	49,000	8.72	68.41
F 59	3 ½ φ = 0.589	2 ⅜ φ = 0.22	5.125	0.937	1.623	4.618	16.22	1.116	727	1315	54,300	9.50	74.61
*F 64	2 ½ φ 1 ½ □ = 0.642	2 ⅜ φ = 0.22	5.125	0.937	1.681	4.606	16.43	1.162	782	1400	59,100	10.18	79.60
F 70	2 ½ φ 1 ⅝ φ = 0.699	2 ⅜ φ = 0.22	5.125	0.937	1.737	4.593	16.81	1.205	832	1485	64,200	10.88	84.83
*F 75	3 ½ □ = 0.750	2 ⅜ φ = 0.22	5.125	0.937	1.786	4.586	16.97	1.247	845	1500	65,900	11.48	89.17

*A ⅞ φ is used in place of ½ □ with load capacity adjusted to slight difference in cross sectional area.

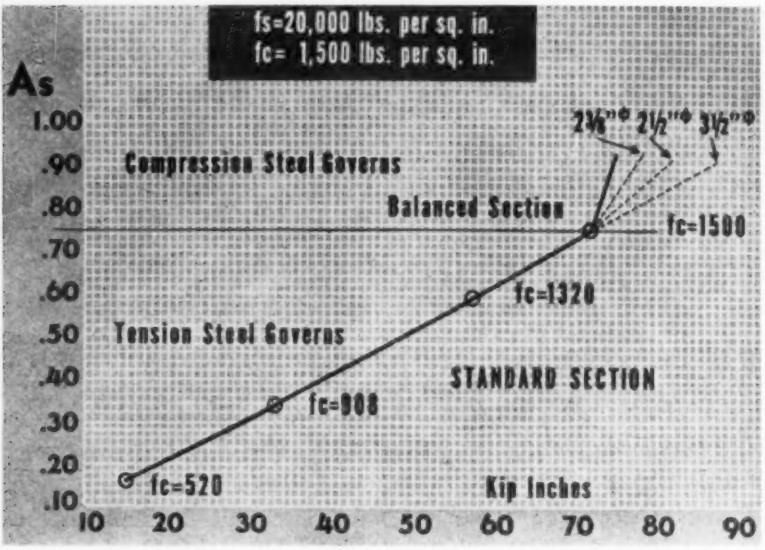

Fig. 3—Safe bending moments for Flexicore slabs

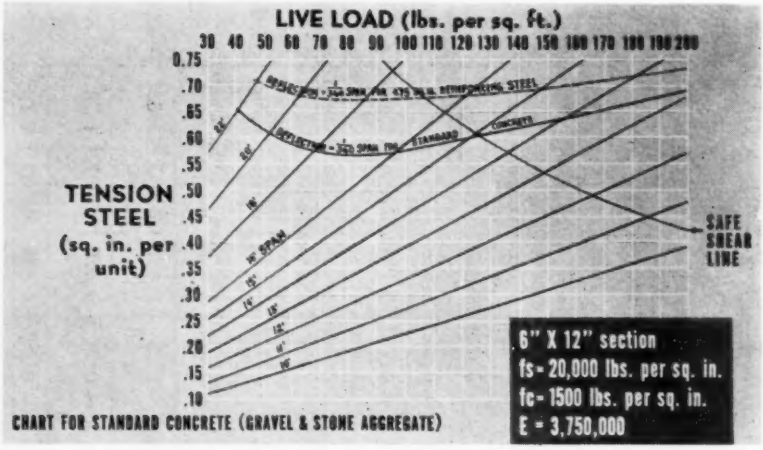

Fig. 4—Live load curves

Table 1 lists the elements for various reinforcements from 0.15 to 0.75 sq in. cross sectional area of tension steel. All the elements of design were calculated and no allowance has been made for the prestressing of the steel. The compression steel, unless for special conditions, is varied only for the length of slab; two ¼-in. rods are used in lengths up to 16 ft 6 in., two 5/16-in. for 16 ft 6 in. to 20 ft lengths, and two ⅜-in. rods for lengths over 20 ft.

Fig. 3 shows the allowable moment in kip-in. for various areas of tension steel. It also shows that up to an area of 0.75 sq in. of tension steel that compression steel is not necessary. The fan lines at the top of the chart show the effect on the curve of various size compression bars. The allowable shear values for the slab without stirrups and with various spacings of both No. 9 wire and $\frac{3}{16}$-in. stirrups are given in Table 2.

On the live load chart in Fig. 4 is shown the allowable loads for various spans and tension steel reinforcements. Also shown on this chart are limits where stirrup reinforcement is required and a curve where calculated deflections equal 1/360 of the span for slabs reinforced with the area of steel shown in the vertical scale of the chart. Above this is another curve showing calculated deflections for a slab with 0.75 sq in. of reinforcement. These data are also available in table form which is handier to use in many cases. These charts comprise the principal data covering the design of this unit.

MANUFACTURE

The typical Flexicore plant consists of a simple rectangular production line through which the forms and slabs travel on conveyors and kiln cars, with a cycle of about one slab every two minutes. Each operation is handled repeatedly by the same workman so that for the most part highly skilled mechanics are not necessary. Fig. 5 shows the plant layout for the different operations. A brief description follows.

The form consists of two side rails, a bottom, and two bulkheads, one stationary and the other adjustable in inches. Alternate bars are attached to the opposing side rails. These when meshed are locked to the opposite rail by a key bolt.

TABLE 2—ALLOWABLE SHEAR—STANDARD 6 IN. x 12 IN. FLOOR SLABS

v = 112.5 psi; f_s = 16,000 psi.

		Shear V, lb
Without stirrups		1230
No. 9 wire	4 in. spacing	1368
	3 in. spacing	1414
	2 in. spacing	1506
$\frac{3}{16}$-in. rod	4 in. spacing	1451
	3 in. spacing	1525
	2 in. spacing	1672

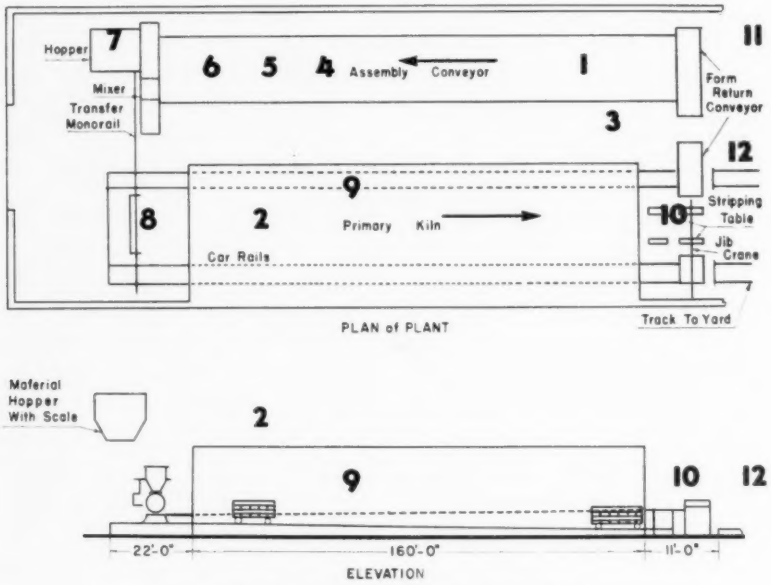

Fig. 5—Layout of typical Flexicore plant

1. Form assembly
2. Top rod and stirrup fabrication
3. Tension rod fabrication
4. Steel insertion
5. Steel prestressing
6. Tube insertion and inflation
7. Pouring operation
8. Screeding
9. Kiln, and tube removal
10. Stripping
11. Cutting as required
12. Inspection and marking

The forms are cleaned and oiled with a mixture of one part castor oil to three parts of fuel oil. Proper care in this operation will prevent formation of bubble holes in the finished ceilings.

Reinforcing steel is purchased in coils. It is straightened and cut to length readily with proper equipment. This is an important feature of economy in both labor and material costs. The top cage is assembled by welding the stirrups to the two top bars. Bolts, $\frac{1}{2}$ or $\frac{5}{8}$ in. in diameter are butt welded to the ends of the bottom steel. These are used in conjunction with sleeve nuts extending through the bulkheads which are tightened by a torque wrench to apply a uniform stress around 20,000 psi to the tension steel. (Fig. 6).

The Flexicore tubes are then placed by simply pushing them through the near bulkhead into the form and out through the far bulkhead. They are inflated so that they will just fill the hole in the bulkhead. The tube shortens as it expands and lengthens when deflated. (Fig. 7).

The form is pulled off the end of the assembly conveyor onto three high frequency vibrators which allow a fairly harsh mix to be used. A hose positioner is used to hold down the tubes and separate them

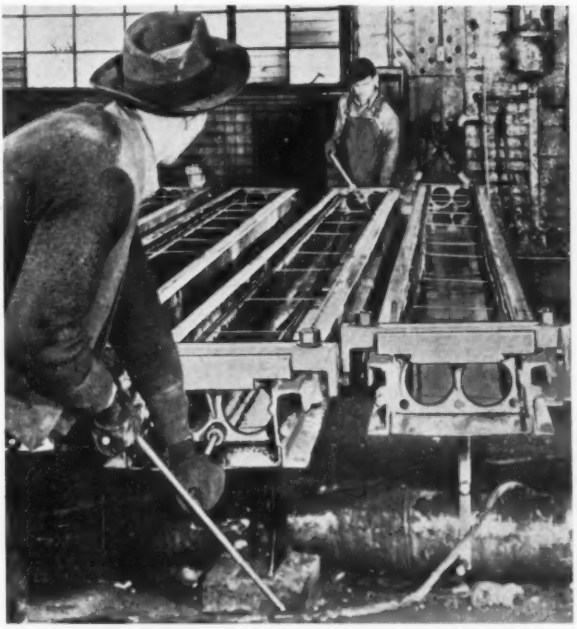

Fig. 6—Prestressing bottom steel with torque wrench. Note reinforcing cage and bulkheads in forms.

Fig. 7—Inflating tubes after they are placed in the forms

during pouring and vibration—otherwise the tubes would float. The pouring apparatus, shown in Fig. 8, consists of a long metal drum with a trough on top and a moving car on the top of the trough to spread the concrete. The drum is slowly rotated and revolves about one-third of a turn to fill a form. The concrete comes out in a layer about 1¼ in. thick and pours all along the slab at the same time. In this way the entire length of the slab is poured at one time, filling the form quickly. The filled form is roughly screeded and then picked up by a traveling hoist and carried to the kiln car. Here the concrete is hand floated to give a true surface without waves and a slab of uniform thickness.

After twenty forms have been set on the kiln car, they are pulled into the kiln on carefully aligned tracks to insure that the slabs will not have any wind in them due to the form being in a twisted position while curing.

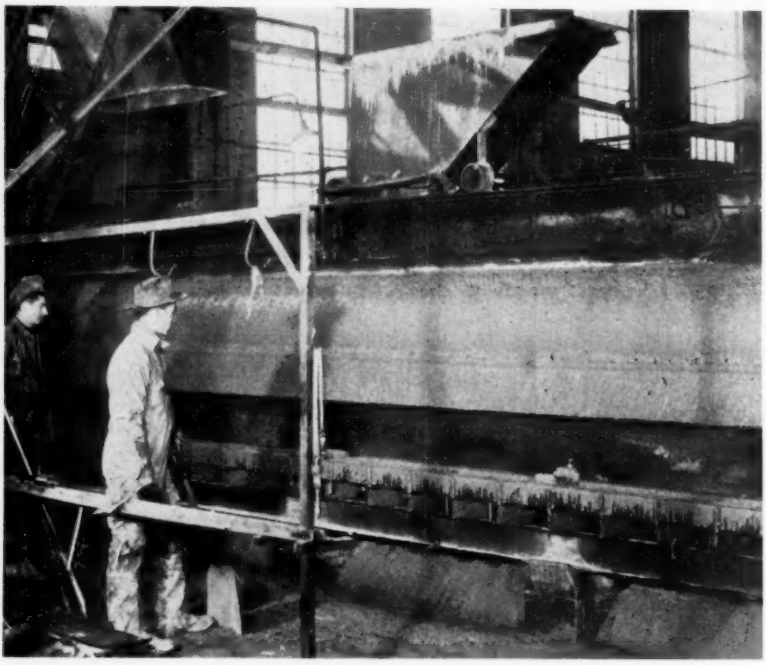

Fig. 8—Pouring apparatus. Form to be filled is at bottom of photograph. Note regularly spaced hose positioners resting on top of form. Hopper is at upper left.

Hot steam pipe radiators between the tracks on the kiln floor give the best control in curing. Water is sprayed on the hot coils to maintain the necessary humidity. Temperatures for a single shift plant range from 80 F in the daytime to about 140 F at night.

The Flexicore tubes are deflated and removed as soon as the concrete has set enough to hold its shape, usually in two to four hours. Tubes are then ready for reuse.

The first step in the stripping operation is knocking out the key bolts and removing the bulkheads. The form and slab are turned upside down and placed on an intermediate table; the hoist then moves over and picks up a previously stripped slab and places it in a bundle for storage. As soon as the stripped slab is out of the way, the one on the intermediate table is pulled forward to the main stripping table where the remainder of the form is removed by peeling off the sides in a simple operation with an ordinary cant hook (Fig. 9). The operation is managed so that the sides transposed by turning the form over can be brought into proper position, without lifting, when reassembled. The bottom pan is taken off by lifting one end. On long forms a "puller" is used

Fig. 9—Stripping forms. Hoist in background is about to place form and slab on intermediate table.

to decrease the possibility of bending the pan and thus causing it to form an untrue slab surface when used again.

The slab is inspected at this point. It must be structurally sound, and have a true and smooth ceiling surface. Slabs are marked to identify the area of the steel rods which reinforce them.

When the plant is run on a one shift operation, forms are stripped about twenty-two hours after pouring at which time concrete strength is about 2,200 psi.

A small percentage of the slabs are cut with an abrasive saw, usually to take care of last minute changes in plan or unforeseen conditions. Cutting is done after curing has been completed. Bending tests of sawed units compare favorably with full length units. No bond failures due to bolt ends being removed when the slab is cut have been observed.

TESTS

Five standard Flexicore slabs made by the Illinois-Wisconsin Concrete Pipe and Tile Co., Beloit, Wis. were loaded as simple beams and tested to destruction by Kurt F. Wendt at the Materials Laboratory, University of Wisconsin in February 1941. A summary of the test data is shown in Table 3.* All slabs were loaded at the center of the span and failed by overstressing the steel in tension; there was no evidence of compressive failure in any case. Except for Slab 5 there was no evidence of shear cracks. In Slab 5 a diagonal crack started from the

TABLE 3—SUMMARY OF LOAD TEST DATA

Slab No.	Span, ft	Design load, lb per sq ft	Deflection at design load, in.	1/360 span, in.	Failure load, lb per sq ft	Ratio, failure load / design load
1	22	40	0.24	0.733	152.7	3.82
2	15	60	0.080	0.500	237	3.95
3	15	60	0.074	0.500	229	3.82
4	4	60	< 0.005	0.133	2400	40
5	2½	60	< 0.005	0.083	6370	106

*Complete test data are available from Institute office at cost of reproduction.

TABLE 4—DEFLECTION OF KEYED SLABS

	Deflections, in.				
	Slab 1	Slab 2	Slab 3	Slab 4	Slab 5
A	5/8	5/8	5/8	5/8	5/8
B	3/4	3/4	3/4	3/4	13/16
C	5/16	3/8	3/8	3/8	5/16
Total	27/16	28/16	28/16	28/16	28/16

A = deflection due to weight of slab.
B = additional deflection due to uniform applied load of 100 lb per sq ft on all slabs.
C = additional deflection due to load on center slab of 250 lb per sq ft.

bottom of the beam about 8 in. from the center just as ultimate load was reached.

Additional tests in May 1942 by George E. Large* at the Engineering Experiment Station, The Ohio State University, on slabs made by Price Brothers Co. of Dayton showed deflection recovery after removal of test load to be well within the limits prescribed by the ACI Code. In these tests the slabs were also loaded to failure.

In tests reported by P. F. Griffenhagen and E. F. Ronnenberg*, structural engineers, it was shown that part of the load on a single slab would be transferred to adjoining slabs by the grouted keys between. Five slabs were grouted into a unit and tested. Resulting deflections shown in Table 4 indicate good load transferability.

Puncture tests were run with a 3/4-in. diameter pin bearing. An average of ten tests gave a value of 4100 lb required to puncture a slab.

Compressive tests on the unit gave the following results:

Flat—(an average of 5 tests)—839 psi.

On edge—(an average of 3 tests)—858 psi.

A test on sound transmission through the slab showed an average reduction of 38.6 decibels in the frequency range of 129 to 2070. It is much quieter than wood construction.

Tests for insulating values showed that with sand and gravel aggregates the C factor was 1.25 and with a lightweight aggregate it was 0.89.

ERECTION

After an order is taken a simple job layout is made, in which every slab is detailed as to position, span and reinforcement. These are submitted for approval by the purchaser. After approval the same details are used for manufacturing and later by the erection superintendent. To secure a good workmanlike job it is, of course, essential that the

*Complete test data are available from Institute office at cost of reproduction.

Fig. 10—Ross carrier delivering a load of slabs

Fig. 11—Placing slabs with a laying cart.

bearing walls have been built true to line and grade. These points are checked before erection operations are begun.

Slabs are delivered either by Ross carrier (Fig. 10) or by truck. Using the Ross carrier the slabs may be delivered several days in advance of erection and stored on the job site. This carrier is about as fast on the road as the average truck and, in addition, can pick up and set down a load quickly.

When delivered by truck, slabs are immediately hoisted into place. Available equipment, local conditions, and job location determine which type of delivery is used.

Since Flexicore slabs weigh 40 lb per ft, it is best to use a crane, hoist truck or similar unit for handling. A truck-mounted hydraulic crane can be set up ready to work in about five minutes.

When erecting, slabs are picked up either with a slab clamp or a belt sling, preferably a clamp as it is safer and much faster.

Slabs are landed on the bearing walls smooth side up and then turned

Fig. 12—Leveling slabs

over by hand. When enough have been placed to form a working deck two wooden 4-in. rollers or 4 x 4's are used to land the remaining slabs. Then they are turned over and put into place with a laying cart (Fig. 11) which is rolled on planks for distributing the wheel load. This method of construction is both simple and fast.

After slabs are all placed, they are aligned by using a pinch-bar, making sure that they are brought together tightly at the bottom to prevent loss of grout and also to give the ceiling a neat appearance.

Leveling is usually accomplished by running pairs of ¾-in. pipes or bars, 8 to 10 ft in length, at right angles to the slabs. Then a nut, bolt and washer are assembled. The nut is slid into the joint between the slabs, one for each joint throughout the length of the leveling bars. Leveling bars are placed on each side of the bolts under the washer and tightened to bring the slabs into alignment on both top and bottom. (Fig. 12). The slabs are then ready for grouting.

Shoring is used, under certain conditions, instead of the clamping method. Where a finished floor is to be poured over slabs, use of shores makes it possible to combine the finished floor and the grouting operations.

Grout is then worked into the joints to form a key, a stiff mixture to plug joints at the end of the slab and a thin mixture to fill the joints. Sometimes it may be necessary to grout the area twice to be sure joints are filled level.

After grouting is finished, the under sides of the slabs are cleaned at the joints. The floor beneath is also cleaned where any grout has fallen through.

Erection in this manner is a simple operation and can be accomplished in practically any kind of weather.

APPLICATIONS

The unit has had a wide variety of uses. Broadly speaking, it can be

Fig. 13—Housing development at Madison, Wis. showing floor slabs in place

used for almost any wall or floor structure where loads not exceeding 50 lb per sq ft on 22-ft span or 200 lb per sq ft on 12 ft span are required. Larger jobs consist principally of housing projects, dormitories, schools, apartments, hospitals and churches, for which the slab is well fitted because of its finished ceiling. Fig. 13 shows its use on a group of buildings in Madison, Wis.

It has been used in some large industrial work (one job containing one-half million sq ft), where it offers better lighting characteristics and minimum maintenance as well as good permanent construction. It has had considerable use on power plant roof decks on which heavy pieces of breeching can be set temporarily during stack erection. Meat packing plants have found the unit well suited to their use because of resistance to damage from high humidity and the ease of steam cleaning when necessary.

The use of this unit in connection with radiant heat using warm air circulating through the cores is expanding rapidly. Both closed and split systems (where air is passed into the room after traversing the floor) are being installed.

The major use has been in many small jobs averaging in size from 1000 to 2500 sq ft. Many of these consist of commercial buildings, about 20 ft wide with masonry walls, where the slab can span from wall to wall making very fast and economical construction. About 25 percent of the production is being used in floors and roofs for residential construction. In the Dayton plant approximately 340 different jobs were processed last year, proof of the versatility of the unit.

Letters from Readers takes over from the former **Job Problems and Practice,** with broader scope, more flexible form, and we hope more adaptability to the thoughtful, busy reader's use, as a forum for opinions, experiences and inquiries within the field of ACI activity. A letter does not need the studied treatment of a major paper to present ideas (the most valuable things we have) for ready access to kindred minds.

Letters from Readers*

BY WAY OF SYNOPSIS

E. L. HOWARD shares information on resistance of concrete floors to acetic and lactic acids—as affected by admixes and surface coatings.

ULF BJUGGREN writes from Stockholm, Sweden about bond in "piano wire concrete."

EIVIND HOGNESTAD, research graduate assistant at the University of Illinois joins the plea for trade name data.

HARRY F. THOMSON and GLENWAY MAXON have more ideas on cause and cure for cement balls in concrete.

F. O. ANDEREGG provides a translation of one European's evaluation of American skill in dam construction.

What to Do About Concrete Deterioration in Food Processing Plants (LR 45-2)†

E. L. HOWARD, testing engineer for Pacific Coast Aggregates, Inc., San Francisco, tells of his experiments with acetic and lactic acids.

I read with pleasure and interest the remarks of Mr. Watts in the September issue of the JOURNAL. During the last few years I have visited many canneries and dairies and can testify that the plea for some action on the problems of acid vs. concrete should certainly be heard and acted upon.

*A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 4, Dec. 1948, *Proceedings*, V. 45. Separate prints are available at 35 cents each. Address 7400 Second Boulevard, Detroit 2, Mich.

†The first letter on this topic was published in the ACI JOURNAL Sept. 1948, *Proc.* V. 45, p. 82.

Several years ago I was asked if certain admixtures in concrete would prevent disintegration of floors in a tomato cannery. To get the answer a series of laboratory and field studies were made. Using a test devised by ourselves, we classified several admixtures whose claims included resistance to acids. The test was a simple one: solution of 5 percent glacial acetic acid or 5 percent lactic acid; immerse floor sample for 24 hours; wash in clear water and brush vigorously with a nail brush, equal to one cycle.

The field observation indicates that about three cycles of this test equal one season of bad conditions in a cannery while one and one-half cycles would equal one season of moderate conditions.

Our tests included both surface treatments and admixtures for concrete: three air entraining agents, one diatomaceous earth, two volcanic ashes, one asphaltic integral admix, six waterproofing agents, calcium chloride base, two surface paints and one cement plaster paint.

The following tables show the results of these tests.

TABLE 1—COMPARATIVE ACID RESISTANCE OF CONCRETE WITH VARIOUS ADMIXES

Admix used in concrete	Number of cycles of test before failure
None	1
No. 1	¾
No. 2	2
No. 3	1½
No. 4	1½
No. 7	No failure after 3 cycles

TABLE 2—COMPARATIVE ACID RESISTANCE OF SURFACE TREATMENTS FOR CONCRETE, AND PLASTER COATS AND FLOOR TOPPINGS WITH VARIOUS ADMIXES

Admix used	Number of cycles of test before failure
None	3
No. 2	4
No. 3	4
No. 5	3
No. 6	5
No. 7	6
No. 8	4
No. 9	2
No. 11	5½
No. 12	3
No. 13	4
Surface Treatments	
No. I	½
No. II	4
No. III	7 (no failure)

Admixes No. 1, 4 and 9 are air-entraining agents. Admix No. 7 is a volcanic ash which resists acid at the expense of surface strength, that is, resistance to wheel abrasion. Its color, however, is very white and has a permanent pearly white luster when used with white cement. No. 11 is a volcanic ash similar to No. 7. These ashes have a cementitious quality when used with portland cement. No. 6 is the asphaltic integral admix.

Of the surface treatments, No. I is a cement-base paint which the acids attack vigorously. No. II is a paint whose composition we do not know. No. III is a rubber-base paint which has proved satisfactory in the laboratory and the field.

We have put small floor patches in canneries and dairies and in one case a complete floor in a small cannery with these materials to coordinate our laboratory tests. We feel our work is not yet conclusive and would be very interested in finding the answer to this perplexing question.

What Will Be Prestress Loss Due to Creep? (LR 45-5)*

Bond in Piano Wire Concrete

ULF BJUGGREN, civil engineer, A B Betongindustri, Stockholm, Sweden

Mr. Anderegg made a question in the September issue about the bond in piano wire concrete. In Sweden piano wire concrete has been produced since 1939 by the company AB Betongindustri, for which I am working and I want to state that we have no difficulties in getting a stabilized and sufficient bond between the wires and the concrete.

The quality of the actual bond depends mainly on the following factors: the quality of the concrete, the working of the concrete in the molds, its strength at release, the type of curing before and after release, the prestress of the concrete, the diameter and the surface properties of the wires and how they are distributed in the cross section. If one single of those mentioned factors is not sufficiently good, the bond might not be satisfactory.

The creep of steel must be taken into consideration when the stretching stress is higher than "the creep point" of the reinforcement. This seems to be actual for some steels used on the European Continent but not for the wires used in the Swedish factory or for some German types of wires.

It is a somewhat startling statement that is given by Dr. Ing. Friesecke and it would be of greatest value to everyone interested in prestressed concrete to have the comments on it by Hoyer himself.

*See also ACI JOURNAL Sept. 1948, *Proc.* V. 45, p. 86.

Why Conceal Trade Names? (LR 45-6)*

EIVIND HOGNESTAD, research graduate assistant at the University of Illinois, also seeks full information.

In reading "Why Conceal Trade Names" in the ACI JOURNAL, October 1948, I found a plea for completeness of reporting in papers and discussions. This plea came from H. H. Edwards, a consulting engineer and user of concrete. The same plea for more explicit information can be made by a student of concrete engineering.

Most knowledge is experience. The work of a student is devoted to absorbtion of knowledge, the experience recorded by those who worked and learned in the past. The great philosophers seriously emphasized "Define your terms", and with the fundamental concepts clear they continued to build their chains of thought. The early engineers did not define their terms—their materials, but they had a good excuse. They did not *know* the significant properties of their materials. It can certainly be excused that a concrete mix was described only as "1:3:3" before Abrams' discovery of the water-cement law. That excuse, however, can not be applied to many reports published in the recent years.

Lately, the writer conducted some tests on the properties of bond between concrete and deformed reinforcing steel. Obviously it was of importance to compare the test results to earlier investigations. Such a comparison could not be made successfully because properties of concrete and bars in earlier bond tests were reported incompletely. Is it excusable to report bond tests after 1940 without information like lug height and spacing? The writer believes it is, under the present practice of engineering report writing. If a "definition of terms" had been stressed and recognized in engineering to the same extent as in many other sciences, however, then it is likely that those lug heights would have been recorded even when they were of no significance to the author at the time he reported his tests. The reports of today are written to convey experience to colleagues of the present time, but more than that, it should never be forgotten that those reports will be read by students of the future. All information that possibly can be significant should be given in a presentation of materials. Even so, a student 20 years from now is likely to find something missing.

A trade name is part of such information. Take the example of an air-entraining admixture. It may be impossible to get detailed information, it may be impossible even to write the author, at a time 20 to 30 years in the future. A manufacturer may also be out of business by then, but a trade name gives one more chance to obtain the infor-

*This topic opened with Harlan H. Edwards' discussion in the ACI JOURNAL Oct 1948, *Proc.* V. 45, p. 177.

mation regarding what "Admixture X" in 1946 actually was. It should be remembered that the student of tomorrow may ask questions we do not consider important today, and he should be left as many keys as possible to sources of information.

Regarding possible misuse of published trade names, the writer recalls the first sentence he heard from an American radio: "The views of the commentator do not necessarily reflect the opinions of the sponsor." It should be possible to report a trade name with a footnote stating, *e.g.*, "This material was obtained from a local dealer in January 1944 and it does not necessarily have the same properties as a material manufactured under the same trade name at an earlier or later date."

Cause and Cure for Cement Balls in Concrete (LR 45-8)*

H. F. Thomson, St. Louis, Mo., advises.

The method of handling materials at a concrete plant is the probable cause of the infrequent occurrence of small balls of cement and sand in fresh concrete. The common procedures in handling materials are (1) mixing complete concrete in a stationary mixer, usually termed "central mixing," and (2) charging materials into a revolving-drum-body (or the equivalent) truck in which the actual mixing is accomplished, termed "truck mixing." For the latter operation two methods of charging are used: either "separate loading" when the sand, aggregate and cement are introduced separately, or "ribbon loading" when all materials are introduced simultaneously. In any case the water should be introduced while the mixer is revolving.

The mixing action in a large stationary mixer is more violent than in a truck-mixer, as the centrifugal speed can be higher. This fact is reflected in the longer mixing time required for a truck-mixer. The violence of mixing in a central mixer mitigates against any likelihood of the above mentioned balls occurring in this type of ready-mixed concrete plant.

In the truck-mixing operation, the balls are not likely to occur when the materials are charged separately into the current types of end-loading drums, as the assembled batch is never quiescent. But with "ribbon-loading," especially if the batch is held for several minutes before charging into the truck-mixer, the damp sand and the cement may be in contact long enough for some cement to stick to some sand; this pellet may accumulate more sand and cement, like a snowball, and the resulting ball may not be broken up during the mixing action. The possibility of balls in a very wet mix is increased if the water is charged rapidly.

*See also ACI Journal Oct. 1948, p. 181; Nov. 1948, p. 261.

Two steps for preventing the occurrence of such balls are suggested—(1) avoiding the contact of sand and cement for an appreciable time before charging the batch into the truck-mixer, and (2) insuring that the water enters the mixer gradually and with such distribution that quantities of cement do not become wet and sticky before being blended with the aggregates. If these steps are not effective then the efficiency of the mixer should be checked, as worn or bent blades may have reduced the actual mixing performance.

GLENWAY MAXON, consulting engineer, Milwaukee, Wis.

No cement balls will form when concrete is mixed in a mixer of three times the volume of the mixed concrete, (this does not refer to mortar or grout,), the mixer being in fair shape in respect to blades, the mixing time being 90 seconds for one cubic yard, with increasingly longer time for larger mixers. Hot cement, wet sand, order of loading the mixer, wet or dry mix, do not cause cement balls in such a mixer with the conditions stated.

Cement balls are most frequently formed when concrete is mixed in transit, or mixed in a transit mixer at the site of discharge. Wet sand is a bad offender, but most of the trouble is caused by overloading. Reduction of the load to less than one-half of the cubic contents and extending the mixing time is the proper answer for transit mixers.

No stationary mixer in fair condition not overloaded, mixes concrete containing cement balls when ordinary precautions are observed. However the art of mixing concrete should have more technical consideration. Experimental work on concrete mixers is badly needed, particularly as the harsher, drier, leaner, air-entraining concretes now being advocated, come into more general use.

Does America Lead in Dam Construction? (LR 45-10)

F. O. ANDEREGG of the John B. Pierce Foundation, Raritan, N. J., sends a translation of editorial comment from a European periodical.

M. R. Ros, of Zurich, Switzerland, who visited the ACI convention in Denver last winter and who spent some time studying dam construction in this country, answers, "Are the Americans ahead of us in dam construction?" in the affirmative in *Technischen Rundschau*, No. 45, 46, 47, 48 and 49 (1947).

He concludes that in dam construction, American methods can be advantageously used in Switzerland. "The Americans honestly try not only to build their dams as cheaply as possible but also as well as

possible, priority being given to quality. Thanks to the experience accumulated by building hundreds of dams, thanks to a natural organizing ability and thanks to a very productive heavy machinery industry, the Americans have produced dams which are museum pieces. The Koering-Johnson preparation installation is the most effective today.

"A wide-spread opinion is found among us, at least among building engineers, that in the U. S. A. work is carried out less carefully than in Switzerland. This concerns a further question, 'Can anyone set up better means for obtaining first-class concrete than the present-day American installations? Can anyone do more in using special cements, in cooling the concrete, in sand blasting the joints, in carefully planning systems of pipe conductors and in sheet consolidation for block joint injection? Is not the application of Hydron and Vacuum concrete a proof of the endeavor to build carefully? Have we ever carried out similar undertakings?'

"If we compare how many investigations the dam constructing people have been carrying out here and in the U. S. A., we suffer badly. It is true that we cannot equal American means, and a quantitative emulation will always be impossible for us. Qualitative advances in dam construction, however, can be made only in relatively costly laboratory studies and field measurements.

"According to the conclusion of the writer, the proper organization of all the details involved in dam building cannot be handled by one person or by a small organization.

"The advances made during the past decade in the U. S. A. in dam construction are very significant and should not be criticized by onlookers.

"Knoxville, and even more Denver, have become centers of information for the whole world. It is no chance happening that American engineers are today building dams in all countries and that Mr. Savage, former chief designing engineer in the Bureau of Reclamation, now serving as their expert, has been consulted on the largest dams in the whole world, for example, in Australia, India, Mexico and Palestine."

Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

Design of space frames with displacement of joints

H. Beer, *Oesterreichische Bauzeitschrift* (Vienna) V. 3, No. 7, July 1948, pp. 103-110

Reviewed by Rudolph Fischl

For this paper the design of space frames is based on statically indeterminate auxiliary force systems. Operating with the displacement of joints reduces the equations for the unknowns at least to the number of possible movements in the hinged system. The Cross method is used to compute the moments. The torsional stiffness is included for the distribution of the moments normal to the respective planes. Two examples illustrate the method.

Silo construction in Denmark

Sv. E. Jacobsen, *Beton-Teknik*, (Copenhagen), V. 14, No. 2, July 1948, pp. 37-46

Reviewed by Eivind Hognestad

The article reports experience gained during the construction of the largest grain silo built in Denmark. The system of silos consisted of 16 circular cells, 32 m tall and with an internal diameter of 6.1 m, arranged in the outline of a square, and 9 smaller cells within the square of larger ones. The total number of 25 cells has a capacity of 11,000 tons of grain. The silos were cast in 17 days with the aid of slip-forms.

Iced concrete

Rock Products, V. 51, No. 3, 1948, p. 149

Highway Research Abstracts Oct. 1948

In order to control the heat generated in the mass concrete of Fort Gibson Dam, Oklahoma, about 500,000 cu yd of iced concrete are being placed by U. S. District Engineers, an ice-making plant having been erected on the site. The temperature of the concrete mix is thus kept within safe limits, *e.g.*, the temperature of a batch was lowered from 95 F to 53 F by adding 162½ lb of ice (2 in. by ½ in. cylindrical pieces) to each cu yd of concrete. This technique eliminates use of cooling liquids through pipes.

New runway laid

Automotive Industries, V. 99, No. 6, Sept. 1948

Highway Research Abstracts Oct. 1948

First stretch of prestressed concrete runway has been installed at Orly Field near Paris, France, by Eugene Freyssinet, who designed and patented the new method.

The strip is 1400 ft long, 200 ft wide and 6 in. thick and is claimed capable of supporting loads equal to those of a conventional runway four times as thick. The experimental installation uses 45 degree diagonal joints making only transverse prestressing necessary. Freyssinet claims his runway can support aircraft three times the weight of any yet built.

Soil cement in house construction

H. Ewaldsen and A. Riber, *Beton-Teknik*, V. 14, No. 1, April 1948, pp. 1-12 Reviewed by Eivind Hognestad

This article gives information in some detail concerning a method of house construction where the walls are constructed from soil stabilized by about 2 sacks of cement per cu yd. Movable forms are used, and the soil-cement is compacted by tamping. Roof and floors are constructed in reinforced concrete. The grading of the soil is controlled. Sand with 8 to 30 percent clay has proved to give satisfactory results. These houses seem to be 10 to 15 percent cheaper than similar houses made from bricks or concrete.

Old and new methods for proportioning of cross sections

A. Aas-Jakobsen, *Betong*, (Stockholm), V. 33, No. 2, 1948, pp. 41-74 Reviewed by Eivind Hognestad

The author presents a thorough critical review of the fundamental concepts of structural design. The two important methods of design, the elastic and the plastic theory, are compared theoretically; and it is shown that the results obtained are verified by tests reported in the literature. For reinforced concrete a combination of the two theories gives rather simple formulas, and a good estimate of the most economical percentage of reinforcement in concrete members is made possible. The combined method also gives simple expressions for additional moments in slender compression members.

Symposium on concrete

P. A. Bech, T. G. Brandtzaeg, Kirsten Friis, S. W. Stephensen and Sven Thaulow *Teknisk Ukeblad*, (Oslo), V. 94, No. 44, 1947, pp. 598-627 Reviewed by Eivind Hognestad

This series of seven articles give a brief consideration of various aspects of present Norwegian concrete practice.

1. The first article gives general information on the proportioning of concrete. The influence of the constituents of the concrete on properties like strength and workability are considered, as well as control of concrete manufacture.

2. The increasing importance of concrete in the Norwegian construction industry is described. Future possibilities of lightweight and prestressed concrete are emphasized.

3. A general review of Norwegian literature on the use of concrete in hydraulic structures is presented. Difficulties pertaining to concrete in contact with fresh and sea water are discussed. Air-entraining agents are mentioned.

4. Air-entraining concrete has been used to some extent in a few dams. Some tests and practical experience are reported.

5. A short review of the development of the concrete pipe and block industry is presented. The manufacturing processes are discussed, especially the factors relating to the permeability and durability of concrete pipes.

6. The sixth article gives a general review of methods of surface finishing. The importance of good workmanship and proper curing is emphasized.

7. The author describes construction of concrete highways in Norway, and he points out how much cheaper the general repair of the highways after the war would have been if concrete pavements had been more extensively used in the past. Compared to other pavements, the concrete has stood up very well.

War-damaged reinforced concrete constructions and their repairs

F. PONGRATZ, *Oesterreichische Bauzeitschrift* V. 3, No. 1-3, January-March 1948, pp. 28-34 Reviewed by RUDOLPH FISCHL

Nine pictures illustrate the description of war damages on factories and other structures in and near Vienna. The economic situation there requires repair of even severely damaged structures rather than rebuilding.

Careful studies of the size and intensity of destructions were carried out, and cooperation of the consulting engineer and contractor succeeded in building the repaired parts monolithic with the undamaged construction. Some of the repaired structures have been in use for almost one year and no cracks have been visible, not even under vibrating loads.

Insulated precast concrete wall panels prove economical substitute for brick

CHARLES H. TOPPING, *Civil Engineering*, V. 18, No. 9, Sept. 1948, pp. 42-45 Reviewed by J. R. SHANK

The du Pont company used precast concrete-gypsum tile units for the outer walls of its nylon plant at Chattanooga, Tenn., at a cost comparable with brick. The walls were 7 in. thick, a 4-in. core of gypsum block between equal concrete facings. Wire mesh was used in each facing and the only other reinforcing was a light perimeter bar in the inside slab. Anchors for inserts were used for handling and erection.

The walls were poured inner face down on asphalt-impregnated kraft paper on a concrete slab. This inner face slab was placed rather dry using a vibrator mounted on 10 steel tired wheelbarrow wheels. The mesh was embedded by this contrivance. The vacuum process of the Vacuum Concrete Company was used on the upper or inside face of this slab. Pre-wetted gypsum tile were then laid on this processed slab and the exterior slab was poured as for the first, and vacuum processed. After the processing a finish coat of grout approximately ¼ in. thick was hard steel troweled on the slab to bring the wall to finished thickness. The edges were round tooled. This steel troweled surface was given a final textured finish with a hair push broom.

Forms were removed and the panels were lifted to upright position the morning after pouring by means of vacuum pads and bolts in inserts and carried to the curing racks where they were water cured with blankets for seven days. The panels averaged 80 sq ft in area though some of them were as large as 200 sq ft. They weighed 65 lb per sq ft. Under ideal conditions one crew placed 2700 sq ft in one day.

Refinements in casting produced unexpected dimensional accuracy with no error as large as ¼ in. A ¼-in. space was maintained between adjoining faces for the sake of appearance.

These slabs were erected on structural steel. Shelf angles were welded at the time of erection to carry these slabs, though the accuracy of dimension was such as to have permitted them to be shop welded.

E.S.L. Bibliography No. 2 (bibliography on prestressed reinforced concrete)

Compiled by The Engineering Societies Library, 29 West 29th Street, New York 18, N. Y., 1948, $4.00

This 25-page mimeographed compilation includes 190 references (both foreign and domestic) to books and periodical literature on the subject in a period of 62 years. The

most recent reference is dated June 24, 1948; the earliest cites United States Patent 339, 296 issued April 6, 1886 to P. H. Jackson for "forming artificial-stone or concrete arches". The first part of the periodical lists 14 references to books; the balance of the references are to periodical articles published largely since 1930. Bibliograhpy may be purchased from the Engineering Societies Library (address above).

Modern design practice and rigid controls produce sound concrete in Ross Dam

H. F. FAULKNER, *Civil Engineering*, V. 18, No. 19, Sept. 1948, pp. 26-31

Reviewed by J. R. SHANK

Ross Dam, the farthest upstream of three dams on the Skagit River in Washington, was originally planned and designed to be built in three stages to elevations 1365, 1615, and 1728 ft. The lowest foundation is at 1075 ft and its extreme base thickness after the construction of the second stage is 200 ft. The second stage construction which produced a crest length of 1264 ft and a storage capacity of 1,400,000 acre-ft is described in part in this paper. Final construction is expected to thicken the dam on its downstream face where key recesses have been provided and raise the crest to 1750 ft and the capacity to 3,400,000 acre-ft.

Two concretes, Class A and Class B, were used; Class A, one barrel of cement per cu yd averaging 4828 psi; and Class B, 1.3 barrels per cu yd averaging 6639 psi at 28 days. The modulus of elasticity was in excess of 5,000,000 psi for both. All of the aggregate was taken from Skagit River bars and was composed of more or less rough, rounded structurally sound particles. The cement was Type II conforming with A.S.T.M. specification C150-41, coming from a mill on the Skagit River 50 miles below the dam. Although the cement displayed strength fluctuations which undoubtedly contributed to variations in the strength of concrete cylinders, it averaged high in quality being low in alumina, magnesia, as well as alkalies.

Many trial mixes were made resulting in mixes containing 6 in. maximum size aggregate and sands of 2 90 to 3.00 fineness modulus having 18 percent passing the 48 mesh screen and 5 percent passing the 100 mesh. The water-cement ratio by weight was 0.54 for the Class A concrete and 0.51 for Class B. The mixing was 2½ minutes in 4-cu yd mixers. Batching was automatic by weight, and no air-entraining agents were used.

Concrete was transported by car for 100 ft, dumped in 8 cu yd bottom-dump buckets and then traveled by cableway. Two-man Chicago pneumatic vibrators were used to place layers 20 in. thick.

Surfaces for bond were prepared by cutting off about ¼ in. of the surface with high pressure air and water jets. Later the surfaces were washed with air and water two or three times. Not until 72 hours had elapsed after placement was a covering pour permitted. Dirty surfaces over seven days old were sandblasted. A ½-in. layer of 1:2.6 bonding grout was broomed into the old surface.

Internal cooling was used and temperatures were not allowed over 90 F, in contrast with the first stage of construction where no cooling pipes were used and temperatures did not run over 100 F. After setting, concrete was continuously hosed on all horizontal surfaces and sprayed through perforated pipes on all vertical surfaces until covered with fresh concrete regardless of the time interval.

Systematic inspection was carried out at all times. Competitions were developed among mixing plant operators and inspectors. Errors were bases for ridicule. A high *esprit de corps* was maintained. Inspectors were given to understand that they were constructors and not merely critics. The writer emphasizes the importance of responsible and continuous on-the-job inspection.

ACI News Letter

Vol. 20 No. 4 JOURNAL of the AMERICAN CONCRETE INSTITUTE December 1948

PREPAKT REPORTS . . .

. . . ON WORK IN PROGRESS

During September, PREPAKT materials and methods were being used in the construction or rehabilitation of more than 100 structures in the United States and Canada. Typical applications in progress at that time included:

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ACI News Letter Contents

December 1948

On the cover—New York City skyline and waterfront after dark. Recognizing the many big-city attractions for the out-of-town visitor, no technical sessions are being scheduled at night. See p. ii (front of book) for the program as it is now shaping up.

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1948 ACI awards will be shared by eight

Wason Medals and the ACI Construction Practice Award for 1948 will go to Frank H. Jackson and Harold Allen; R. C. Mielenz, K. T. Greene and E. J. Benton; and Raymond E. Davis, E. Clinton Jansen and W. T. Neelands. This announcement comes from the ACI Board of Direction following approval of the report of the Awards Committee. Formal presentation of awards will be made by President Blanks at the 45th Annual ACI Convention Luncheon in New York February 24.

Frank H. Jackson and Harold Allen have been awarded the Wason Medal for the "most meritorious paper" of the year,

Concrete Pavements on the German Autobahnen

which appeared in the ACI JOUNAL, June 1948.

Richard C. Mielenz, Kenneth T. Greene and Elton J. Benton are recipients of the Wason Medal for "noteworthy research", reported in

Chemical Test for Reactivity of Aggregates with Cement Alkalies; Chemical Processes in Cement-Aggregate Reaction

which appeared in the ACI JOURNAL, November 1947.

These Wason awards, established in 1917 by the late Leonard C. Wason, past president of the Institute, consist of an inscribed bronze medal and certificate to each awardee. One award is made for the "most meritorious paper" published in the latest volume of *Proceedings* of the American Concrete Institute; the other, the research medal,

is awarded for original work or discovery whose subject matter falls within the scope of Institute activity, and which has been within the year the subject of an Institute paper.

Another trio of authors, Raymond E. Davis, E. Clinton Jansen and W. T. Neelands, will receive the ACI Construction Practice Award for the paper

Restoration of Barker Dam

which was published in the ACI JOURNAL, April 1948 following a brief presentation at the Denver convention. Messrs. Davis, Jansen and Neelands will share the token of the award—United States savings bonds, Series E, with maturity value $300. This award for a paper of outstanding merit on concrete construction practice was established by the Institute in 1944 to recognize the construction man whose resourcefulness bridges the gap between the job "on paper" and the solid fact of a completed structure.

The Award Winners

Frank H. Jackson and Harold Allen —Wason Medalists

Frank H. Jackson, ACI vice president and an Institute member since 1924, is a native of Washington, D. C. He attended Ohio State University and in 1905 was appointed to the Public Roads Administration (then the Bureau of Public Roads) where he has served successively as assistant engineer of tests, engineer of tests, senior engineer of tests and more recently in his present capacity—principal engineer of tests. He was in direct charge of the non-bituminous road materials laboratory of the Bureau from 1915 to 1924, and in general charge of non-bituminous road materials investigation (including concrete) since 1924.

Mr. Jackson has been author or co-author of seven JOURNAL papers, has served on numerous ACI technical committees as well as publications, advisory and program committees; he is now a member of the Technical Activities Committee. Active in the American Society for Testing Materials, he was recently awarded honorary membership in that society. He is also a member of the American Association of State Highway Officials.

Harold Allen was graduated in civil engineering at the University of Colorado and received his master's degree at Kansas State College. From 1921 to 1927 he was engaged in concrete research and in the testing of highway materials at Kansas State College. From 1927 to 1937 he was materials engineer for the State Highway Commission of Kansas. Since 1937 he has been a materials engineer for the Public Roads Administration and has devoted most of his time to soils research. At present he is in charge of the subgrade section of the Division of Physical Research. As chairman of Highway Research Board committees, he has been active in the investigation of the warping of concrete pavements and in the pumping at joints between concrete pavement slabs.

Richard C. Mielenz, Kenneth T. Greene and Elton J. Benton— Wason Medalists

Richard C. Mielenz, co-author of three JOURNAL papers and Institute member since 1945, attended the University of California, Berkeley, where he received his Ph. D. in 1940. He was employed as geologist by the Standard Oil Co. of California in the San Joaquin Valley from 1939 to 1941. He then went to Denver to join the petrographic laboratory of the Bureau of Reclamation where he is now head. He has engaged in application of petrography and geology to engineering

problems, particularly that of aggregate selection. As evidenced by the nature of his ACI contributions, he has been engaged also in basic research on construction materials, particularly the problem of cement aggregate reaction.

Kenneth T. Greene attended the New York State College of Ceramics, Alfred, N. Y. receiving a bachelor's degree in glass technology and engineering in 1935. Upon completion of graduate work in ceramics at Rutgers University, New Brunswick, N. J., he received a Ph.D. degree in 1940. He was associated with the Portland Cement Association Fellowship at the National Bureau of Standards as research chemist from 1940 to 1945. Dr. Greene joined the staff of the petrographic laboratory of the Bureau of Reclamation in 1945 and is engaged in the application of chemical and petrographic methods to problems involving cement, concrete and other engineering materials. In 1947 he became an ACI member.

Elton J. Benton attended Western State College of Colorado and graduated in 1938 with a degree in chemistry. In 1942 he joined the staff of the U. S. Bureau of Reclamation and participated in the physical-chemical investigations of concrete aggregates begun at that time. During 1944 and 1945 Mr. Benton served in the U. S. Navy as technician in electronics, seeing action at Okinawa. He returned to the petrographic laboratory early in 1946, and since that time has been engaged in continuing research on aggregates, concrete, cement admixtures and other engineering materials.

Raymond E. Davis, E. Clinton Jansen and W. T. Neelands—Construction Practice Award

Raymond E. Davis, past president of the Institute, author or co-author of more than a dozen ACI papers, twice a Wason medalist (1931, 1934), scarcely needs a long introduction to regular JOURNAL readers. He retired from the ACI Board of Direction last February after 18 years of continuous service with that body.

Graduating from the University of Maine in 1911, Professor Davis went to the University of Illinois where he taught for 7 years, receiving his M.S. degree there in 1916. After a year as associate professor of civil engineering at the University of Nebraska, he went in 1920 to the University of California where he is now professor of civil engineering and director of the Engineering Materials Laboratory.

An ACI member since 1926, he is also affiliated with the A.S.C.E., A.S.T.M., Sigma Xi, Tau Beta Pi, Chi Epsilon and Delta Sigma Chi. He is a fellow of the American Association for the Advancement of Science.

E. Clinton Jansen served from 1907 to 1916 as hydraulic engineer with the Central Colorado Power Co. and the Colorado Power Co. From 1916 to 1924 he was chief engineer for the Colorado Power Co. In 1924 he became hydraulic engineer of the Public Service Co. of Colorado, in which capacity he was associated with the work on Barker Dam. Mr. Jansen retired from this position, and from active work in general, August 1, 1947.

In addition to holding ACI membership since 1944, he is a life member of the American Society of Civil Engineers and an associate member of the Institution of Civil Engineers (London).

W. T. Neelands, resident engineer at Barker Dam from March 1946 to September 1947, has been an ACI member since 1943. From 1936 to 1940 he was inspector, general construction, for the U. S. Bureau of Reclamation at Grand Coulee Dam, Wash., and from 1940 to March 1943 served in a similar capacity at Shasta Dam, Redding, Calif. He then joined Barrett and Hilp Contractors as assistant

concrete engineer during the construction of 20 concrete barges at San Francisco, Calif. He was co-author (with E. B. Hanson, Jr.) of "The Effect of Curing Condition on Compressive, Tensile and Flexural Strength of Concrete Containing Haydite Aggregate" (ACI JOURNAL, November 1944, *Proc.* V. 41, p. 105) which was based on tests of lightweight concrete made during this ship-building program.

In 1944 Mr. Neelands worked briefly as ship supervisor for Permanente Metals Corp. Since termination of his work at Barker Dam, he has been materials engineer on the Clark Hill Project of the Department of the Army, Corps of Engineers, Savannah District, at Augusta, Georgia.

Pumice and Flexicore films available

ACI member Jack Counts of Pumice Aggregate Sales Corp., Albuquerque, N. M., has announced the availability of "an excellent color movie on the production and uses of pumice aggregate and pumice concrete that might be of interest to the ACI members". Those interested in showing the film locally may address inquiries to Mr. Counts.

"The Story of Flexicore", a sound-slide film which presents in great pictorial detail the information covered by this month's paper (p. 325), is available for showing by private groups. Requests for this film and the accompanying recording should be made to The Flexicore Co., Dayton, Ohio.

Inspection trips for New York convention-goers

ACI members going to New York February 23-25 may want an extra day or two to see several construction projects of more than usual interest which can be visited with a minimum of effort and time.

Among these outstanding projects is the Brooklyn-Queens Connecting Highway which features an unusual three-level cantilever structure to carry this highway through downtown Brooklyn. The 10.5-mile connecting highway, of which the three level structure is a half mile section, has been classed as an expressway since it will carry all kinds of traffic.

Two outstanding examples of reinforced concrete frame and slab band construction may be seen in the New York City Housing Authority project: Lillian Wald Houses in downtown New York at East 6th Street and Franklin Roosevelt Drive; and the New York Life Insurance Company housing development at Fresh Meadows, Flushing, Long Island. The Lillian Wald construction consists of 12 buildings of 11 and 14 stories and Fresh Meadows has two 13-story units and 132 small buildings.

A preview of New Jersey's Route 100 which will be an all purpose highway connecting the major arteries in the metropolitan area between New Brunswick and the George Washington Bridge can be seen in the first dual-dual road to be built in the East. This extends some 3 miles from Elizabeth to Newark Airport. It has been paved with air-entraining concrete, and light-reflecting concrete curbs, both sloping and vertical, were used throughout. An important safety feature is the construction of an emergency lane for breakdowns on each of the separate roadways. This 8-lane highway handles an average of over 60,000 vehicles daily with peak loads greater than 100,000 in 24 hours.

Now under construction is a 3-mile extension from the Newark Airport to the Pulaski Skyway where the present road is being dualized to provide 8-lanes of travel which will be elevated.

One other project warranting attention is the New York International Airport at Idlewild, in Queens, Long Island. Seven runways have now been completed for an approximate total of 1,250,000 sq yd of concrete. Ultimate plans call for additional runways and tremendous areas of apron pavement.

Who's Who This Month

C. P. Siess and N. M. Newmark

From the University of Illinois comes the Siess-Newmark contribution, "Rational Analysis and Design of Two-Way Concrete Slabs", (p. 272) based on Professor Siess' Ph.D. thesis. Offering a new slab design procedure which may replace part of the current ACI Building Code, the paper will be briefed for presentation for open discussion at the New York convention in February.

Special research assistant professor of theoretical and applied mechanics at the University of Illinois, Chester Paul Siess was born in Alexandria, La. in 1916. Following graduation in civil engineering at Louisiana State University, he worked as soils engineer in the testing laboratory of the Louisiana Highway Commission. In 1937 he received an appointment as special research graduate assistant in theoretical and applied mechanics at the University of Illinois where he worked on a cooperative investigation of reinforced concrete highway bridges and received his M. S. in 1939.

Following employment by the Chicago Department of Subways and Superhighways and the New York Central Railroad, he returned to the University of Illinois in 1941 as a special research associate, with assignment to further bridge investigation. In 1945 he was appointed to his present university position.

Co-author of several University of Illinois Engineering Experiment Station publications, Mr. Siess is affiliated with ACI, A.S.T.M., Sigma Xi, Tau Beta Pi, Phi Kappa Phi, Omicron Delta Kappa and Phi Eta Sigma. He is a member of the Society for Experimental Stress Analysis, an associate of the Highway Research Board and an associate member A.S.C.E. He is active on A.S.T.M. committees, and for ACI he is a member of Committee 321 and chairman of Committee 215, Fatigue of Concrete.

Nathan M. Newmark, research professor of civil engineering at Illinois, is also active in ACI work as a member of the Technical Activities Committee, and chairman of Committee 321, Design of Reinforced Concrete Slabs. He is currently a candidate for a three-year term on the Institute's Board of Direction.

Born in 1910, he graduated with special honors in civil engineering at Rutgers University in 1930, and then went to the University of Illinois as a research graduate assistant in civil engineering. Since then he has been continuously on the university staff, and was appointed to his present position last year. Concurrently he served as consultant for several army and navy units during the war years, and has also done consulting work in structural design and machine analysis.

Doctor Newmark has written for publication a number of technical papers on such topics as structural analysis and design (see "Structural and Economic Studies of Monolithic Concrete Walls for Dwellings", ACI JOURNAL May-June 1935, *Proc.* V. 31, p. 478), bridge floors, theory of elasticity, soil mechanics and fatigue of metals. In addition to his ACI affiliation, he is a member of the A.S.C.E., A.S.M.E., the Institute of Aeronautical Sciences and the International Association of Bridge and Structural Engineers. He is also a member of A.S.T.M., A.R.E.A., Society for Experimental Stress Analysis, Phi Beta Kappa, Tau Beta Pi, Sigma Xi and Chi Epsilon.

E. W. Scripture, Jr. and C. H. Sakryd

E. W. Scripture, Jr. and C. H. Sakryd contribute knowledge of floor resistance to chemical attack in "Corrosion Resist-

ance Tests of Concrete Floors—With and Without Metallic Aggregate", p. 317. Mr. Scripture, author of a 1936 paper "Metallic Aggregate in Concrete Floors", was introduced last month as co-author of "Influence of Size Grading of Sand on Air Entrainment".

C. H. Sakryd graduated in chemistry at Case Institute of Technology in 1913. He was employed as chief chemist of the analytical laboratory and later as research chemist at Graselli Chemical Co. until 1922. While with the Graselli company, he was co-author of a zinc chloride table entitled "Specific Gravity of Aqueous Solutions at $\frac{60^\circ}{60^\circ}$ F" now used as a standard in chemistry handbooks.

From 1922 to 1930 he did specialized work in spray drying of various chemical compounds and food products, particularly vegetables and soaps. The Lever Brothers Co. retained Mr. Sakryd as spray drying expert in patent litigation on soap from 1935 to 1940. Since 1941 he has been engaged in plant control and research at the research laboratories of the Master Builders Co., Cleveland.

Gayle B. Price

"Flexicore Precast Floor and Roof Slabs", p. 325, is Gayle B. Price's first contribution to ACI JOURNAL pages. This amply illustrated paper is based on the sound-slide film shown before a Denver convention audience in February.

A graduate of Antioch College, Mr. Price is treasurer of the Price Brothers Co., Dayton, Ohio, and vice president of Townsend and Bottum Co. of Ann Arbor, Mich. He has been an ACI member since 1943, and serves on the Institute's Committee 711, Precast Floor Systems for Houses.

The Price Brothers Co. entered the precast concrete field in the early thirties with the manufacture of concrete sewer pipe and now also produces prestressed pressure pipe, concrete blocks, and the Flexicore slabs described by Mr. Price. Production of these slabs on a national scale is licensed by Flexicore Company, Inc., a Price subsidiary.

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Cemenstone correction

The footnote to "Cemenstone Precast Construction." p. 193, November JOURNAL, should read in part: "...Title No. 45-11 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 3, Nov. 1948, *Proceedings* V. 45". Discussion of this paper closes March 1, 1949, not Jan. 1, 1948 as indicated.

New Members

Seventy-seven applications for membership were received in October and approved by the Board of Direction; of these 38 were Individual, 10 Junior, 29 Student.

Illinois was top membership state for the month providing 24 new member applicants. Florida was second with 5 applicants.

Our membership total November 1, 1948 reached 4343.

Alabama

Cornelius, William E., (Indiv.) 364 Starnes Park, East Gadsden, Ala.
Sawyer, Herbert Allen, Jr., (Indiv.) Box 1844, University, Ala.

Arkansas

Johnson, John P., (Indiv.) 1007 B West Grand Ave., Hot Springs, Ark.

California

Bradshaw, Richard R., (Indiv.) 7551 Melrose Ave., Los Angeles 46, Calif.
Fong, Hayward, (Jr.) 1233 W. Ninth St., Los Angeles 15, Calif.
O'Neil, Hugh M., (Indiv.) 610 16th St., Oakland 12, Calif.
Scofield, George N., (Indiv.) 2071 Laura Ave., Huntington Park, Calif.

District of Columbia

Frazier, Richard, (Jr.) 6th & Emerson Sts. N.E., Washington 17, D. C.

Florida

Birdsall, Robert J., (Jr.) P. O. Box 661, Daytona Beach, Fla.
Channing, Jules P., (Indiv.) 1575 Washington Ave., Miami Beach, Fla.
Orgill, W. A., (Indiv.) 5600 N. W. 7th Ave., Miami, Fla.
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Gould, Ralph W., (St.) Forrest, Ill.
Hamant, Vincent Francis, (St.) 310 W. Elm St., Urbana, Ill.
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Hodge, George McFarland, Jr., (St.) 605 N. State St., Champaign, Ill.
Hooper, William T. F., Jr.,(Indiv.) Dept. of Civil Engineering, Northwestern University, Evanston, Ill.
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Nelson, Norman A., (St.) Std. Terr. J-41-A, Champaign, Ill.
O'Leary, Cornelius F., (Indiv.) 1400 Sedwick St., Chicago 10, Ill.
Rootberg, Albert, (Indiv.) 5557 W. Jackson Blvd., Chicago 44, Ill.
Seder, Alan Grantham, (St.) 903 W. Oregon St., Urbana, Ill.
Unsin, Roy E., (St.) 102 E. Healey St., Champaign, Ill.
Unteed, Donald O., (St.) 145 Evanslawn Ave., Aurora, Ill.
Varley, Henry Harold, (St.) RR-1, Beecher City, Ill.
Wharton, William V., (St.) 506 S. 2nd St., Fairbury, Ill.

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Senour, Vernon K., (St.) Bourbon, Ind.
Tsai, I. S., (St.) 116 Marstellar St., E. Lafayette, Ind.

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Pierson, Daniel Penton, (Jr.) 327 S. Elm Ave., Webster Groves 19, Mo.

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Hepburn, Donald, (Indiv.) c/o Kellex Corp., 12th Floor, 2 Lafayette St., New York, N. Y.

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Washington

Baker, W. McNeill, (Jr.) 606 E. Indiana Ave., Spokane 13, Wash.

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Richards, J. D., (St.) Queen's University, Kingston, Ont., Canada

Schwenger, William C., (Indiv.) 114 Prospect St. S., Hamilton, Ont., Canada

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Allen, Luis G. Solano, (St.) Apartado 171, Cartago, Costa Rica

Alvarado, Guillermo E., (Indiv.) Sn Vicente de Moravia, Costa Rica, C. A.

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Parajon, Luis, (St.) Ave. Almendares 13, Alturas de Almendares, Havana, Cuba

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Positions and Projects — ACI Members

Dan H. Pletta

Dan H. Pletta, ACI member since 1931, recently became head of the department of applied mechanics at Virginia Polytechnic Institute. He has been on that school's faculty since 1932, most recently acting as professor of applied mechanics. In World War II Professor Pletta served as a major in the mechanics department at the U. S. Military Academy at West Point and with the Philadelphia Ordnance District.

E. P. Newhard

E. P. Newhard, general chemist at the Nazareth plant of the Pennsylvania-Dixie Cement Corp., has been appointed operating manager, succeeding Carl J. Lofstedt. Following early periods of employment with the U. S. Geological Survey and the Atlas Portland Cement Co., Mr. Newhard was an assistant and later chief chemist of the Giant Portland Cement Company at Egypt, Penn., and Norfolk, Va., from 1910 to 1918. During the following 6 years he was chief chemist of the Clinchfield Portland Cement Co. at Kingsport, Tenn., and from 1926 to 1932 was chief chemist of the southern division of the Pennsylvania-Dixie Cement Corp. In 1944 he was appointed general chemist in charge of chemical control and research at Nazareth, Penn. Mr. Newhard has been an ACI member since 1946.

George B. Sowers

George B. Sowers, ACI member of Cleveland, has been reappointed by Gov. Thomas J. Herbert as Ohio public works director. His term of appointment extends until July 31, 1949.

Lawrence company appoints Pope

Lawrence Portland Cement Co., of which James H. Ackerman is president, has announced the appointment of Herbert W. Pope as assistant to the president. Mr. Pope's headquarters will be at 150 Broadway, New York 7. The Lawrence company has been an ACI corporation member since 1916.

Harmer E. Davis heads transport institute

Harmer E. Davis, professor of civil engineering at the University of California, has been appointed acting head of the recently organized Institute of Transportation and Traffic Engineering at that school. Immediate objectives of the new institute, established by a 1947 act of the California legislature, relate to development and improvement of highways, although the enabling bill is broad enough so that other modes of transportation may ultimately receive attention.

As an agency functioning under and in conjunction with the university, the primary function of the Institute of Transportation and Traffic Engineering is education and the advancement of knowledge of the field. With respect to the educational function, three general types or programs of training are being planned: (1) training of a graduate and professional type, (2) various phases of in-service training, and (3) the dissemination of factual information to public agencies and groups other than those composed of technical engineers. With respect to the collegiate training, the intent is to make available over a period of years, trained engineers in sufficient number to man the proposed construction program, and to develop students in the field with a broad outlook on the problems of modern transportation. On the undergraduate level, the institute, through its teaching and research staff, will strengthen and augment the present undergraduate transportation option in civil engineering. On the graduate level, courses of study are being established leading to advanced professional degrees.

Other members of the professional staff of the new institute are: Dr. Donald S. Berry, assistant director and professor of transportation engineering; Ralph A. Moyer, professor of civil engineering and research engineer for the institute; Cecil J. Van Til, lecturer in transportation engineering; and Fred N. Finn, lecturer and administrative assistant for the institute.

M. A. Craven heads New Zealand Portland Cement Association

M. A. Craven, Institute member since 1943, has been selected to direct the activities of the newly formed New Zealand Portland Cement Association. Headquarters will be established in Wellington as soon as satisfactory office space can be arranged. Until that time temporary headquarters will be maintained in the Certified Concrete Ltd. building at 151 Grant Rd., Wellington N. 1., New Zealand. Contributors to the association will include all existing cement companies in New Zealand and the association's activities will be on a smaller scale but much the same as those of the Portland Cement Association in the United States.

Frank S. Altman retires

Frank S. Altman, ACI member since 1935, has retired from the post of district engineer of the Portland Cement Association's Minneapolis office after 28 years of service with that association. He entered P.C.A. employ in 1920 as field engineer in Kansas and had held the Minneapolis position since 1922. He is succeeded by Fred R. McComb who has been field engineer in Indiana.

Harry C. Shields

Harry C. Shields, Institute member since 1926, has left his position with the Marquette Cement Manufacturing Co. to become eastern manager of the construction division of the Presstite Engineering Co., St. Louis, Mo.

Alvin W. Hicks

Alvin W. Hicks, service engineer in the Memphis office of the Marquette Cement Manufacturing Co., has been appointed sales manager in the St. Louis office. He will be in charge of sales in Missouri, southern Illinois and southern Indiana. Mr. Hicks is a concrete specialist of wide experience and a member of the Institute. He received his engineering training at the University of Texas and subsequently was employed in the engineering divisions of the Missouri Pacific Railroad, Arkansas State Highway Department, and U. S. Engineers.

Albyn Mackintosh

ACI member Albyn Mackintosh, consulting engineer in Los Angeles, Calif., has been named an honorary member of the Concrete Masonry Manufacturers Association of Southern California.

Peter J. Doanides

Institute member Peter J. Doanides, second vice president of the Matrad Corp., New York, has gone to Johannesburg, South Africa to become managing director and chief engineer of Concrete Development Corporation Ltd. This company

is a newly formed subsidiary of Matrad, which is the exclusive licensee of the vacuum concrete process in the Union of South Africa.

E. M. Fleming succeeded by Hall

John F. Hall has been appointed district engineer in charge of the New York office of the Portland Cement Association, to succeed ACI member E. M. Fleming, whose request for retirement was granted November 1.

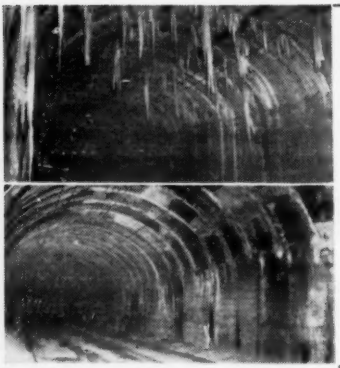
Mr. Fleming, who has held managerial positions with the association in the eastern area for the past 13 years, entered its employ in April 1926 in the capacity of field engineer in Indiana. Two years later he was transferred to the association's general office, and in March 1929 was appointed manager of the Highways and Municipal Bureau, in which capacity he served for six years. He was appointed district manager of the New York office in 1935, and served continuously in that capacity except for an eight-year period from January 1937 to June 1945 when he also was manager of eastern offices.

Honor Roll

February 1 to October 31, 1948

Prof. Newlin D. Morgan, University of Illinois, mounts to the top of the current Honor Roll with credit for 33 new Members for the period ended October 31, 1948. His son, Prof. Newlin D. Morgan Jr., is a close second with 30 credits. Honor-Rollers in California, Massachusetts, Costa Rica and elsewhere — note that this honor roll tally will end February 1—less than three months remain to be registered.

Newlin D. Morgan (Ill.).....33
Newlin D. Morgan, Jr. (Wyo.).....30
Raymond E. Davis (Calif.).....15
Thomas E. Stanton (Calif.).....14
Howard Simpson (Mass.).....10
Leon Venegas (Costa Rica).....7
James A. McCarthy (Ind.)..... 6½
Raphael Callejas H. (Guatemala).....6
Harrison F. Gonnerman (Ill.).....6
Henry L. Kennedy (Mass.).....6
Oliver G. Julian (Mass.)..... 5
Wm. H. Thoman (Colo.).....4½
E. I. Fiesenheiser (Ill.).....4
R. H. Sherlock (Mich.).....4
Lewis H. Tuthill (Colo.).....4
Phil M. Ferguson (Texas).....3½
Elmo C. Higginson (Colo.).....3½
J. Antonio Thomen (Dominican Republic).....3½
Bailey Tremper (Wash.).....3½
G. E. Troxell (Calif.).....3½
Harold S. Carter (Utah).....3
Robert W. Freeman (Calif.).....3
Ray C. Giddings (Calif.).....3
Myle J. Holley Jr. (Mass.).....3
George E. Large (Ohio).....3
A. E. Mackney (Canada).....3
F. N. Menefee (Mich.).....3
Rene S. Pulido y Morales (Cuba).....3
Robert Morris (Colo.).....3
James M. Polatty (Ga.).....3
Richard A. Roberts (Calif.).....3
Joseph J. Waddell (Calif.).....3
Alberto Dovali Jaime (Mexico).....2½
Thomas C. Kavanagh (Pa.).....2½
Frank E. Richart (Ill.).....2½
John A. Ruhling (Ill.).....2½
John C. Sprague (Ga.).....2½
H. D. Sullivan (Wash.).....2½
Stanton Walker (D. C.).....2½
Hugh Barnes (Calif.).....2
F. Campus (Belgium).....2
John K. Class (Pa.).....2
Moses D. Cohan (N. Y.).....2
W. S. Cottingham (Wis.).....2
Jacob J. Creskoff (Pa.).....2
H. J. Gilkey (Iowa).....2
Ernst Gruenwald (N. Y.).....2
Edward L. Howard, Jr. (Calif.).....2
R. R. Kaufman (Ohio).....2
John C. King (Ohio).....2
William E. Lumb (Canada).....2
Robert B. B. Moorman (Mo.).....2
Hugh E. Odor (Wash.).....2
George L. Otterson (Ark.).....2
Walter H. Price (Colo.).....2
Manuel Ray Rivero (Cuba).....2
John W. Robison (Colo.).....2
Georges Ronai (Turkey).....2
Thomas C. Shedd (Ill.).....2
C. H. Scholer (Kans.).....2
E. Copeland Snelgrove (England).....2
J. Neil Thompson (Texas).....2
E. W. Thorson (Colo.).....2



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Eugene Mirabelli (Mass.).....1½
W. E. Parker (Canada).....1½
Dean Peabody, Jr. (Mass.).....1½
Jerome M. Raphael (Colo.).....1½
Clarence Rawhouser (Colo.).....1½
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Calvin T. Watts (La.).....1½
Harold E. Wessman (Wash.).....1½
Ray A. Young (Wash.).....1½
Jerome O. Ackerman.....1
J. B. Alexander.....1
W. H. Allison.....1
A. Amirikian.....1
Boyd G. Anderson.....1
Edwin C. Anderson.....1
J. A. Bakker.....1
George E. Barnes.....1
Paul H. Barnes.....1
S. B. Barnes.....1
F. Barona.....1
Frank H. Beinhauer.....1
Rene L. Bertin.....1
Robert F. Blanks.....1
A. J. Boase.....1
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John Butkus.....1
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A. N. Dutta Choudhury.....1
C. O. Christy.....1
Miles N. Clair.....1
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A. Finlayson.....1
T. H. Fleming.....1
R. E. Fowle.....1
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R. R. Gore.....1
Emil A. Gramstorff.....1
Eugene Guillard.....1
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Paul Weidlinger....1
J. M. Wells....1
H. Roger Williams....1
Merle D Wilson....1

The following credits are, in each instance "50-50" with another member.

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Oscar Vega Arguelles
Manuel A. Arosemena
S. Balasubranmanyam
A. Allan Bates
E. W. Bauman
C. T. Bishop
Petrus Manzano Bonilla
P. G. Bowie
Harold W. Brewer
A. R. Brickler
Boyd S. Brooks
L. Rees Brooks
Sterling Lowe Bugg
B. W. Burns
Jose Luis Casas
Thomas F. Chace
Wilbur H. Chamberlin
Richard de Charms
R. V. Chate
J. E. Christiansen
James H. H. Chun
H. W. Chutter
Emil W. Colli
F. Coppen
Rolland Cravens
J. M. Crom
R. A. Crysler
Philip Davis
Charles W. De Groff
John G. Dempsey
R. F. Dierking
J. B. Dolsen
J. E. Dunn
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John N. Eckle
Harlan H. Edwards
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F. L. Ehasz
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William E. Feidt
Paul Fischer
Rudolph Fischl
E. M. Fleming
Frederick E. W. Fowler
Jaichand Hirachand Gandhi
Leo V. Garrity
Carl V. Gezelius
Luis Alberto Truque Gurdian
J. R. G. Hanlon
Bernard Harrison
Frank W. Haws
H. L. Henson
A. E. Hiscox
W. C. Hodgkins
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G. W. Holcomb
H. N. Howe
Walter L. Huber
J. W. Hubler
Manuel Castro Huerta
Robert B. Hyslop
C. L. N. Iyengar
Orville E. Jack
Frank H. Jackson
M. E. James
Bruce Jameyson
E. B. Johansen
Harry D. Jumper
J. W. Kelly
Thomas M. Kelly
John J. Kennedy Jr.
K. J. Krawczyk
H. M. Larmour
Robert E. Lee
L. G. Lenhardt
John F. Long
Wallace A. Marsh
Hubert F. McDone ll
Juan B. McKay
Callix E. Miller, Jr.
Hugh Montgomery
Ray M. Moorhead
E. J. Mueller
A. E. Myers
Israel Narrow
Elmer H. Oechale
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Emil H. Praeger
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Fredric Rusche
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Joseph J. Shideler
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Journal of Southeastern Research

A new scientific journal covering engineering research progress in eleven southeastern states will begin publication in January. The *Journal of Southeastern Research*, to be published by the Southeastern Research Institute of Atlanta, will report the activities and accomplishments of Southeastern laboratories and "serve as a conference table for research leaders".

Editorial and news columns of the *Journal* will cover activities in Alabama, Arkansas, Florida, Georgia, Louisiana, Mississippi, North Carolina, South Carolina, Tennessee, Texas and Virginia. Technical contributions from these states will be given preference, although papers from other states will be used if they have special value to the Southeast. Editor of the new publication will be H. McKinley Conway, Jr., a graduate of the Georgia Institute of Technology, and former assistant to the director of research, National Advisory Committee for Aeronautics, Washington, D. C.

N. J. Manniche

A November visitor at ACI headquarters in Detroit was N. J. Manniche, managing director of Manniche and Hartmann Ltd., civil engineers and contractors of Copenhagen, Denmark. Mr. Manniche is making a coast-to-coast tour of the United States, studying in particular prestressed and precast concrete construction.

Prestressed reinforced concrete bibliography issued

"Bibliography on Prestressed Reinforced Concrete", ESL Bibliography No. 2, has been released by the Engineering Societies Library, 29 West 39th Street, New York 18, N. Y. The 25-page mimeographed publication lists some 190 books and periodicals from world-wide sources. Annotated references cover all aspects of the subject, including theory, design, construction practice and applications treated in published literature of the past 60 years. Most of these publications have appeared since 1930. Price of the bibliography is $4.00.

Revised A.S.T.M. specifications for air-entraining cement

The American Society for Testing Materials has announced changes in its Specification C175-47T for air-entraining portland cement. Addition of a new Type IIIA cement — high-early-strength air-entraining — has made necessary a change in the "scope" paragraph at the beginning of the specification. The new scope is as follows:

"Scope — These specifications cover three types of air-entraining portland cement for use where air entrainment is desired as follows: Type IA—for use in general concrete construction when the special properties specified for Types IIA and IIIA are not required; Type IIA—for use in general concrete construction exposed to moderate sulfate action, or where moderate heat of hydration is required; Type IIIA—for use when high early strength is required."

This "scope" was approved at the June 1948 meeting of Committee C-1 on Cement and has subsequently been approved by letter ballot of the society.

NOTICE!

Copies of original books and magazine articles reviewed in the Current Reviews section of the JOURNAL are **not** available from ACI. The footnote formerly published with that section indicated that separate prints of *the reviews section itself* are available, not separate prints of the individual articles reviewed, as some readers have assumed.

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SYNOPSES of recent ACI Papers and Reports

Institute papers of this JOURNAL Volume 20 are currently available at prices indicated. Please order by title and title number.

RECOMMENDED PRACTICE FOR WINTER CONCRETING METHODS (ACI 604-48).....45-1

Price 50 cents.

REPORT of COMMITTEE 604—Sept. 1948, pp. 1-20, (V. 45) in special covers

Supersedes 44-13.

This ACI standard establishes methods of cold-weather concreting for thin sections and mass concrete. Heating of materials, accelerators and anti-freezes, curing and temperature records during curing, subgrade (or base) preparation, protective coverings during curing, and form removal are discussed for both types of job, and preferred methods are indicated. An appendix entry outlines objectives of the special winter methods with background material which indicates the "why" of some of the recommended practices. Charts in the appendix indicate effect of curing temperature on concrete strength, and a list of 135 selected references to periodical literature on winter concreting methods is included.

RELATIVE STRENGTHS OF PORTLAND CEMENT MORTAR IN BENDING UNDER VARIOUS LOADING CONDITIONS.....45-2

Price 35 cents.

JACOB PORTER FRANKEL—Sept. 1948, pp. 21-32 (V. 45)

Assuming the behavior of standard mortar to be similar to that of plain concrete, tests were performed on 99 small mortar beams under sixth-, third- and center-point loading to verify the applicability of the statistical theory of the strength of brittle materials to concrete structures. The theory, as first developed by Weibull, is briefly analyzed, compared to the methods used by Tucker, and finally applied to the tests at hand. Agreement between the theoretical and experimental findings is so close as to warrant future testing on plain concrete specimens.

TERRAZZO AS AFFECTED BY CLEANING MATERIALS.....45-3

Price 35 cents.

D. W. KESSLER—Sept. 1948, pp. 33-40 (V. 45)

Effect of cleaning materials on terrazzo was studied using solutions of the following detergents: soda ash, trisodium phosphate and synthetic sulfonate. Seventy-six different aggregates were used in preparing terrazzo disks which were moist cured 3 months and then surface ground to form a shallow dish. Detergent was allowed to stand in dish 30 minutes before rinsing and drying (at 105 C). This testing cycle was repeated until deterioration tendencies were established. Soda ash solution proved much more destructive than the trisodium phosphate; however, up to the time of writing (after 850 cycles) the synthetic sulfonate had caused no failure.

TRANSPORTING READY-MIXED CONCRETE IN OPEN DUMP TRUCKS.....45-4

Price 35 cents.

R. A. BURMEISTER—Sept. 1948, pp. 41-56 (V. 45).

Milwaukee experience in hauling air-entraining concrete in open dump truck bodies is recorded. Slump. air content. specific weight, compressive strength and workability tests were made on concrete before and after a trial run over bumpy pavement. Some similar tests were made on concrete being placed in a Milwaukee street after open truck transportation. The author concludes that open trucks for hauling ready-mixed air-entraining concrete are satisfactory when air content is between 3 and 6 percent (5 to 6 sacks cement per cu yd) and hauling time is 45 minutes or less, provided that aggregate moisture, grading and slump are carefully controlled.

A STUDY OF ALKALI-AGGREGATE REACTIVITY BY MEANS OF MORTAR BAR EXPANSIONS.....45-5

Price 35 cents.

T. M. KELLY, L. SCHUMAN and F. B. HORNIBROOK—Sept. 1948, pp. 57—80 (V. 45)

Many types of aggregate were combined in varying amounts and sizes with high- and low-alkali cements and formed into 1 x 1 x 10-in. mortar bars. The bars were stored either at 70 F or at 100 F and their expansions measured at ages ranging from 1 month to 4 years.

In combination with high-alkali cements, opal, opaline chert and a siliceous dolomitic limestone were found to cause greatest expansion. Certain aggregates containing volcanic glasses and some natural sands and gravel also caused excessive expansion; with one exception, these sands contained small amounts of opal.

Greatly delayed expansion resulted with the very fine sizes of opal, particularly in combination with high-soda cement. Similar behavior resulted with minus No. 81 size opal and low-alkali cement with either Na_2SO_4 or K_2SO_4 additions.

Materials such as dehydrated kaolin, soda feldspar, magnesium fluosilicate, acetic acid and calcium hydroxide added in small amounts as correctives were ineffective. However, diatomaceous earth in sufficient quantity as a cement replacement eliminated expansion.

REINFORCED CONCRETE WALL AND COLUMN FOOTINGS PART I.....45-6a

Price 60 cents.

FRANK E. RICHART—Oct. 1948, pp. 97-128 (V. 45)

PART II.....45-6b

Price 35 cents.

FRANK E. RICHART—Nov. 1948, pp. 237-260 (V. 45)

This paper presents, in two parts, a report on an experimental investigation of reinforced concrete footings. It represents the first extensive study of the subject since the well-known work of Talbot in 1913. In the meantime there have been many developments in materials, as well as in design methods.

Important developments from these tests include the following: (a) Definite proof that the tensile and bond resistance depend upon the bending moment and shear found by statics by consideration of the full applied load, and not 85 percent thereof, as assumed in current building codes; (b) Hooked ends of bars showed no particular advantage in bond resistance over straight bars, particularly when deformed bars of new improved types were used; (c) Hooked bars produced very little effect on resistance to diagonal tension, much less than is commonly assumed; (d) Welded mats proved particularly effective in resisting end slip of bars; (e) Footing caps or piers are effective in reducing the amount of reinforcement and in increasing the load capacity of footings; and (f) Diagonal tension seems to be the point of weakness in current design practice. The factor of safety of thin footings in this respect appears greater than in thick footings, and is generally greater in rectangular than in square footings when the conventional methods of computation are used.

In these tests, major emphasis has been placed on the isolated column footing. Principal attention has been

Current ACI Standards

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Minimum Standard Requirements for Precast Concrete Floor Units (ACI 711-46)
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4 pages: 35 cents per copy

Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-48)
A publication of large format, bound to lie flat.
55 pages: $3.00 per copy ($1.75 to ACI Members)

Recommended Practice for Winter Concreting Methods (ACI 604-48)
20 pages in covers: 50 cents per copy (40 cents to ACI Members)

Recent Proposed ACI Standards

The Nature of Portland Cement Paints and Proposed Recommended Practice for Their Application to Concrete Surfaces
Reported by Committee 616 as information and for discussion only. 20 pages, 35 cents per copy (Reprint from ACI JOURNAL, June 1942)

Proposed Recommended Stresses for Unreinforced Concrete
Reported by Committee 322 as information and for discussion only. 4 pages, 35 cents per copy. (Reprint from ACI JOURNAL, Nov. 1942)

given to the resistance of footings to failure by bond, diagonal tension and tension in the steel: Test footings were designed to produce these various types of failure. In addition, studies were made of the behavior of rectangular footings and footings having intermediate capitals or piers.

The tests featured measurements of tensile and compressive strains, deflections and slippage of bars. In some of the tests three grades of concrete were used, in others, five types of reinforcing bar were employed. In most of the tests, the footings were 7 ft square. They were supported on a bed of steel springs and tested in a large testing machine.

This paper merely attempts to present and interpret the test results, and does not offer recommendations as to possible changes in design procedures at this time.

LATTICE ANALOGY IN CONCRETE DESIGN.....45-7

Price 35 cents.

DOUGLAS McHENRY—Oct. 1948, pp. 129-140 (V. 45)

The lattice analogy is a scheme for solving two-dimensional stress problems in which the involved mathematical methods of the theory of elasticity are replaced by simple computations. The solid section is replaced by an equivalent lattice or framework which may be solved by methods applicable to indeterminate structures. Stress distribution in sections of complex shape and with complex loading may be determined by successive approximations which involve only substitution in simple equations. The method is illustrated by application to the problem of stresses in a deep beam with off-center loading.

STRENGTH OF PRECAST CONCRETE FLOOR JOISTS.....45-8

Price 35 cents.

JACOB FELD—Oct. 1948, pp. 141-148 (V. 45)

The strength and usability of precast concrete floor joists left in the open without any protection for a year, when a housing project was abandoned, were proved by load test on a slab section incorporating the poorest joists. This paper describes the condition of the joists, the load test results and indicates that a greater tolerance can be safely permitted in the visible defects of precast concrete joists.

EFFECT OF ENTRAINED AIR ON CONCRETES MADE WITH SO-CALLED "SAND-GRAVEL" AGGREGATES.....45-9

Price 35 cents.

PAUL KLIEGER—Oct. 1948, pp. 149-164 (V. 45)

Sand-gravel aggregates (maximum particle size $\frac{3}{8}$ in.) used with non-air-entraining cement produce concretes containing from 3 to 4 percent air. The same mixes made with air-entraining cement and the sand-gravel aggregates have air contents from 8 to 13 percent. Effect of this larger amount of entrained air is given in terms of tests of flexural and compressive strength, freezing and thawing resistance, and length changes in varying storage conditions. Effect of "sweetening" the sand-gravel by addition of 30 percent of coarser aggregate is noted.

BURNED SHALE AND EXPANDED SLAG CONCRETES WITH AND WITHOUT AIR-ENTRAINING ADMIXTURE.....45-10

Price 35 cents.

P. H. PETERSEN—Oct. 1948, pp. 165-176 (V. 45)

The physical properties of several lightweight aggregate portland cement concretes made with burned shale or expanded slag were investigated at the National Bureau of Standards. Three grades of concrete were made with each aggregate. Air-entrainment greater than 20 percent is reported for the mixtures leanest in cement, an air-entraining admixture being used to increase the workability of all but the richest concretes. Compressive, flexural and bond strength data are given as well as resistance to heat transfer, rain penetration and water penetration by capillarity. Also included are the coefficients of thermal expansion, shrinkage, and values for change in length due to wetting and drying.

CEMENSTONE PRECAST CONSTRUCTION.....45-11

Price 35 cents.

A. C. GRAFFLIN—Nov. 1948, pp. 193-204 (V. 45)

Standardized precast reinforced concrete structural members and panels point toward economy in fireproof construction of ordinary buildings up to four stories. The method described has been so developed that architects and engineers can select from tables reinforced concrete beams, columns, roof, floor, and wall panels in the same manner as for structural steel. Design is in accordance with the ACI Code and costs compare favorably with similar structures framed of steel and fireproofed with concrete.

SAND GRADING INFLUENCE ON AIR ENTRAINMENT IN CONCRETE.45-12

Price 35 cents.

M. A. CRAVEN—Nov. 1948, pp. 205-216 (V. 45)

Four series of mixes with varying air-entraining agents, cement factors, and sand grading and content were prepared in order to observe sand grading effect on concrete. Graphic record is presented of air content and W/C plotted against fineness modulus of sand; flow and compressive strength plotted against fineness modulus of sand; and air content plotted against percent of No. 30—No. 50 sand and total sand percentage. Generally the percentage of air entrained in concrete increased with decrease in fineness modulus of sand. Quantity of air appears to be a function of the quantity of No. 30—No. 50 sand. Effect of grading and quantity of sand on other properties of fresh and hardened concrete is noted.

INFLUENCE OF SIZE GRADING OF SAND ON AIR ENTRAINMENT.45-13

Price 35 cents.

E. W. SCRIPTURE, Jr., F. B. HORNIBROOK, and D. E. BRYANT—Nov. 1948, pp. 217-228 (V. 45)

Field reports indicated difficulty with certain sands in securing the desired amount of entrained air; this was frequently attributed to the size grading of the sand, particularly to a deficiency in the finer fractions. An experimental investigation was undertaken in view of the paucity of published data on this subject. Mixes of sand and water alone, 1:4 and 1:2 mortars, and concrete mixes were made with and without air-entraining agents and the air contents determined. While size grading of the sand had a very large influence on air entrainment in a mixture of sand and water alone, this effect was smaller in mortars and very small in concrete mixes. It was found that maximum air was entrained by the 28-48 mesh size sand rather than the 48-100 mesh size.

USE OF PREPACKED AGGREGATE CONCRETE IN MAJOR DAM CONSTRUCTION.....45-14

Price 35 cents.

PAUL BAUMANN—Nov. 1948, pp. 229-236 (V. 45)

This paper sets forth the method of construction of a major gravity dam whereby the average temperature of the mass concrete may be controlled within the mean annual and the dry shrinkage reduced to a fraction of that of conventional concrete by means of prepacking of the coarse aggregate to a density corresponding to 30 percent voids; precooling the aggregate; the intrusion of the coarse aggregate by mortar with a cement content of two sacks per cu yd or less; and the internal post-cooling, curing and draining of the mass concrete. (This process is known as the Prepakt method of construction.)

ACI publications in large current demand

Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-48)

Approved as a Standard of the American Concrete Institute at its 44th Annual Convention February 25, 1948. Ratified by letter ballot June 29, 1948. It is of large format, bound to lie flat and presents typical engineering and placing drawings with discussion calling attention to important considerations in designing practice. It was prepared to simplify, speed, and effect standardization in detailing. It is believed to be the only publication of its kind in English. It is meeting wide acclaim among designers, draftsmen and in engineering schools. Price—$3.00, to ACI Members—$1.75.

ACI Standards—1948

202 pages, 6x9 reprinting current ACI Standards: Building Code Requirements for Reinforced Concrete (ACI 318-47); Minimum Standard Requirements for Precast Concrete Floor Units (ACI 711-46); five recommended practices: Use of Metal Supports for Reinforcement (ACI 319-42); Measuring, Mixing and Placing Concrete (ACI 614-42); Design of Concrete Mixes (ACI 613-44); Construction of Concrete Farm Silos (ACI 714-46); Winter Concreting Methods (ACI 604-48); and two specifications: Concrete Pavements and Bases (ACI 617-44) and Cast Stone (ACI 704-44)—nine ACI Standards in one book, $2.00 per copy—to ACI Members, $1.25. The "Detailing Manual" (ACI 315-48) is available separately only at its established price—see above.

Air Entrainment in Concrete (1944)

92 pages of reports of laboratory data and field experience including a 31-page paper by H. F. Gonnerman, "Tests of Concretes Containing Air-entraining Portland Cements or Air-entraining Materials Added to Batch at Mixer," and 61 pages of the contributions of 15 participants in a 1944 ACI Convention Symposium, "Concretes Containing Air-entraining Agents," reprinted (in special covers) from the ACI JOURNAL for June, 1944, $1.25 per copy, 75 cents to members.

Air Entrainment in Concrete—Book 2 (1947)

A 204-page compilation of information on air entrainment published after Book 1 (1944), including: "Field Use of Cement Containing Vinsol Resin", and "Laboratory Studies of Concrete Containing Air-Entraining Admixtures", by Charles E. Wuerpel, "Entrained Air in Concrete", a foreword and 14 short papers presented at the 1946 Convention; and discussion of the symposium, reprinted from ACI JOURNALS for September 1945, Feb., June and Dec. Part 2, 1946. $2.25 per copy, $1.50 to ACI Members.

ACI Manual of Concrete Inspection (July 1941)

This 140-page book (pocket size) is the work of ACI Committee 611, Inspection of Concrete. It sets up what good practice requires of concrete inspectors and a background of information on the "why" of such good practice. Price $1.00—to ACI members 75 cents.

"The Joint Committee Report" (June 1940)

The Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete submitting "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete," represents the ten-year work of the third Joint Committee, consisting of affiliated committees of the American Concrete Institute, American Institute of Architects, American Railway Engineering Association, American Society of Civil Engineers, American Society for Testing Materials, Portland Cement Association. Published June 15, 1940, 140 pages. Price $1.50—to ACI members $1.00.

Reinforced Concrete Design Handbook (Dec. 1939)

This report of ACI Committee 317 is in increasing demand. From the Committee's Foreword: "One of the important objectives of the committee has been to prepare tables covering as large a range of unit stresses as may be met in general practice. A second and equally important aim has been to reduce the design of members under combined bending and axial load to the same simple form as is used in the solution of common flexural problems."—132 pages, price $2.00—$1.00 to ACI members.

For further information about ACI Membership and Publications (including pamphlets presenting synopses of recent ACI papers and reports) address:

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RATIONAL ANALYSIS AND DESIGN OF TWO-WAY CONCRETE SLABS.....45-15

Price 60 cents.

C. P. SIESS and N. M. NEWMARK—Dec. 1948, pp. 272-316 (V. 45)

A new method for the design of two-way building slabs is proposed and its step-by-step development is described. A new moment distribution procedure is used to compute moments in a number of rectangular slabs continuous over rigid beams. Several variables are studied in the analyses and include: the ratio of sides, the effect of discontinuous edges, the torsional stiffness of the beam, various types of loading, and combinations of panels of various sizes and shapes. Certain conclusions are stated regarding the types of loading to be considered and the values of beam torsional stiffness to be assumed in the development of the design procedure.

Moments obtained in the foregoing analyses are modified to take into account the effect of additional variables. First, the slab moments are increased by various amounts as a result of the deflection of the beams. Next, separate coefficients for dead and live load are replaced by combined coefficients for a live load-dead load ratio of 3.0. Then follows a study to determine the effect of discontinuous edges, and this variable is eliminated from the procedure. Finally all slab moments are reduced by 20 percent in recognition of the redistribution of moments at high load. Additional studies are then made of the distribution of moments across the width of the slab and of the moments to be used for the design of the beams to complete the development of the design procedure.

A proposed design procedure is presented in specification form and is compared with other procedures of a similar nature and with the moment distribution procedure described in the appendix.

CORROSION RESISTANCE TESTS OF CONCRETE FLOORS—WITH AND WITHOUT METALLIC AGGREGATE.....45-16

Price 35 cents.

E. W. SCRIPTURE, JR. and C. H. SAKRYD—Dec. 1948, pp. 317-324 (V. 45)

An attempt has been made to determine the conditions under which concrete floors can be expected to give good service. Resistance to abrasion was determined on specimens with and without metallic aggregate after exposure to typical reagents selected from the groups described by the Joint Committee as having corrosive effects of an intermediate nature between no attack and severe attack. On the basis of the experimental results the reagents in the Joint Committee report have been classified in relation to anticipated service of concrete floors.

FLEXICORE PRECAST FLOOR AND ROOF SLABS.....45-17

Price 35 cents.

GAYLE B. PRICE—Dec. 1948, pp. 325-340 (V. 45).

This paper describes briefly the design, manufacture, erection procedures and use of Flexicore slabs in precast concrete floor and roof systems. The Flexicore slab has a standard cross section 12 in. wide and 6 in. thick, lightened by two holes 4⅝ in. in diameter running the full length of the slab which is made in inch variations of length up to a maximum of 22 ft 6 in. Reinforcement consists of two bars in the top of the slab for "handling" stresses and two or three prestressed tension bars at the bottom to take care of bending moment.

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Separate prints of current ACI papers and reports are available at the prices listed in the adjoining synopses. Send your requests for these and other information to:

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DISCUSSION

Discussion closes January 1, 1949

Sept. Jl. '48

Recommended Practice for Winter Concreting Methods (ACI 604-48)—Report of Commitee 604

Relative Strengths of Portland Cement Mortar in Bending Under Various Loading Conditions—Jacob Porter Frankel

Terrazzo as Affected by Cleaning Materials—D. W. Kessler

Transporting Ready-Mixed Concrete in Open Dump Trucks—R. A. Burmeister

A Study of Alkali-Aggregate Reactivity by Means of Mortar Bar Expansions—T. M. Kelly, L. Schuman, F. B. Hornibrook

Discussion closes February 1, 1949

Oct. Jl. '48

Lattice Analogy in Concrete Design—Douglas McHenry

Strength of Precast Concrete Floor Joists—Jacob Feld

Effect of Entrained Air on Concretes Made With So-Called "Sand-Gravel" Aggregates—Paul Klieger

Burned Shale and Expanded Slag Concretes With and Without Air-Entraining Admixture—P. H. Petersen

Discussion closes March 1, 1949

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Sand Grading Influence on Air Entrainment in Concrete—M. A. Craven

Influence of Size Grading of Sand on Air Entrainment—E. W. Scripture, Jr., F. B. Hornibrook and D. E. Bryant

Use of Prepacked Aggregate Concrete in Major Dam Construction—Paul Baumann

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Dec. Jl. '48

Rational Analysis and Design of Two-Way Concrete Slabs—C. P. Siess and N. M. Newmark

Corrosion Resistance Tests of Concrete Floors—With and Without Metallic Aggregate—E. W. Scripture, Jr. and C. H. Sakryd

Flexicore Floor and Roof Slabs—Gayle B. Price

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(ACI PROCEEDINGS Vol. 45)

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ACI history and objectives. The Institute was founded 1905, incorporated in the District of Columbia in 1906 as The National Association of Cement Users, the name changed 1913 by charter amendment. It was re-incorporated August 8, 1945 with these objects: "to organize the efforts of its members for a non-profit public service in gathering, correlating and disseminating information for the improvement of the design, construction, manufacture, use and maintenance of concrete products and structures.

Papers, committee reports and other contributions to the **Proceedings** pages of this Journal, as submitted in furtherance of Institute objectives, are, if published, accepted on the sole responsibility of their authors. Institute authority attaches only to standards adopted as provided in the By-Laws. All manuscripts prepared with a view to Journal publication should be submitted in triplicate, addressed: Secretary, Technical Activities Committee, American Concrete Institute, New Center Building, Detroit 2, Michigan.

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Journal subscription price $10.00 per year payable in advance. Members receive the Journal as one of the considerations of dues payment. The Journal is issued monthly, January to June and September to December (the volume year September to June being completed by title page, contents, indexes and closing discussion in Part 2 of the subsequent December issue).

Bound volumes 1 to 43 of PROCEEDINGS of the AMERICAN CONCRETE INSTITUTE (1905 to 1947) are for sale as far as available, at prices to be had on inquiry of the Secretary-Treasurer. Special prices apply for members ordering bound volumes in addition to the monthly Journal.

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WEDNESDAY AFTERNOON

Experience with Air-Entraining Concrete in New Jersey by Charles M. Noble, State Highway Engineer.

Control of Scaling of Portland Cement Pavements Resulting from the Use of Chloride Salts by B. D. Tallamy, Superintendent of New York State Department of Public Works.

What Have We Learned about Air-Entraining Concrete? by J. F. Barbee, Engineer, Ohio Department of Highways.

Practices, Experiences and Tests with Air-Entraining Agents in Making Durable Concrete by R. F. Blanks and W. A. Cordon, U. S. Bureau of Reclamation, Denver.

THURSDAY MORNING

Rational Analysis and Design of Two-Way Concrete Slabs by C. P. Siess and N. M. Newmark, University of Illinois.

Reinforced Concrete Wall and Column Footings by Frank E. Richart, University of Illinois.

Experimental Aids in Structural Concrete Design by Robert E. Glover, D. J. Olsen and Carl Zangar, U. S. Bureau of Reclamation, Denver.

THURSDAY NOON

Luncheon—ACI Awards presented by President Blanks

THURSDAY AFTERNOON

Report of Election of Officers

Retiring President's Address

Problems in the production and use of ready-mixed concrete—discussion opened and led by Stanton Walker.

FRIDAY MORNING

Research—Annual open session of Committee 115.

FRIDAY AFTERNOON

Design Details for Architectural Concrete by John J. Hogan, Portland Cement Association, New York.

Construction Practices for Architectural Concrete by E. B. Oberly, Portland Cement Association, New York.

Lightweight Aggregate Concrete by Ralph W. Kluge, Purdue Univ., formerly with National Bureau of Standards, M. M. Sparks, E. C. Tuma, National Bureau of Standards, Washington, D. C.

One more important paper not yet in hand and therefore not identified by title at this time.

*Formerly Hotel Pennsylvania

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is edited by the Secretary of the Technical Activities Committee under the direction of the Committee

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POLICY

It is the policy of the American Concrete Institute to encourage participation by its members and others in the work of extending the knowledge of concrete and reinforced concrete as a basis for improved products and structures.

To this end the Board of Direction has assigned to the Technical Activities Committee the responsibility of selecting for publication such papers, committee reports, discussions and other contributions or parts of such contributions as, in the judgment of the Committee, seem to offer most of value in attaining Institute objectives within space requirements, consistent with budget limitations.

A NEW ACI STANDARD is proposed by Committee 616, G. E. Burnett, Chairman, for the application of portland cement paint to concrete surfaces. The report has been released by the Standards Committee and is published on p. 353. According to rules governing the adoption of standards the report will be presented by the originating committee for adoption at the Convention.

●

Although the Hotel Statler has set aside a large block of rooms for ACI guests, reservations should be made early. Deadline—February 13, 1949. Write direct to Hotel Statler or use the special card recently sent you. ACI cannot assume responsibility for reservations.

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Note carefully the closing date for discussion of papers and reports. Discussion for contents of the Sept. JOURNAL closed Jan. 1, 1949. Closing date for discussion of papers in this issue—May 1, 1949.

Manuscripts of papers, discussions and reports should be sent in triplicate to:

Secretary, Technical Activities Committee

AMERICAN CONCRETE INSTITUTE
NEW CENTER BUILDING, DETROIT 2, MICHIGAN

In ACI Journal next month

"Experimental Aids in Structural Design" is the joint effort of R. E. GLOVER, O. J. OLSEN and CARL ZANGER, which describes equipment used in the analysis and design of indeterminate structures at the Bureau of Reclamation laboratories, Denver, Colorado. Practical applications to typical problems and the interpretation of results are discussed. Mr. Glover will present the highlights at the 45th annual convention.

R. F. BLANKS and W. A. CORDON report Bureau of Reclamation experiences with air-entraining concrete in "Practices, Experiences, and Tests with Air-entraining Agents in Making Durable Concrete". They discuss air-entraining agents and changes in physical properties of concrete resulting from entrained air. A condensed version is scheduled for the New York convention.

GERALD MILSOM draws on his experience with concrete floors for his recommendations on "Concrete Floor Finishing" in the February JOURNAL. Careful supervision and skilled workmanship are rated as the most important factors in producing durable floors.

In "Some Factors Affecting Air Entrainment", E. W. SCRIPTURE, JR., and F. J. LITWINOWICZ give an account of laboratory tests in which studies of various factors in relation to air content were confined to a single variable to eliminate incidental effects. Variables reported on are slump, cement factor, and sand-total aggregate ratio.

"Simplified Design for Eccentrically Loaded Columns" was evolved by P. DEJONGH for reinforced concrete columns subject both to bending and compression. The factors affecting such a condition and derivation of formulas used in this design method are given.

A review of literature on air-entraining concrete is reported by J. F. BARBEE in "What Have We Learned About Air-entraining Concrete?" He points out the effects of entrained air on concrete properties and methods of controlling and using air-entraining concrete.

The 1942 report of Committee 616 filled a need for information on how to use portland cement paints. Published as information and for discussion only it has been in considerable demand for over six years. The present report, a revision and rearrangement of the former report as a recommended practice, is offered for adoption as a standard by the reorganized committee.

Proposed Recommended Practice for the Application of Portland Cement Paint to Concrete Surfaces*

Reported by ACI Committee 616

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SYNOPSIS

This report is a studied revision by a reorganized committee of the previous report of Committee 616 published in June 1942, based upon criticisms of that report plus information from more recent tests and experiences. Included are recommendations for appropriate usage, age of concrete, preparation of surface, and the preparation, application and curing of portland cement paint. Three appendixes discuss composition, manufacture and storage, and general characteristics and factors affecting durability.

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*This report was submitted to letter ballot of the committee which consists of 8 members; 8 members returned their ballots of whom 8 have voted affirmatively and none negatively. It is released by the Standards Committee for publication and discussion with view to consideration for adoption at 45th Annual Convention, New York, Feb. 23-25, 1949.

Title No. 45-18 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 5, *Proceedings* V. 45, and supersedes Title No. 38-30 published June 1942. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than May 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

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INTRODUCTION

1. In recent years, there has been a widening utilization of paint on concrete, both as a decorative and a water repellent treatment. With it, there has developed an increased consciousness of cost, annual as well as initial. The consumer has observed that some paint jobs fail within 2 years and others last four or five times this long. He cannot afford to repaint every 2 or 3 years and wants to know what paint to use and how to get the most out of it.

2. This report has been prepared as a guide toward the realization of economy in painting concrete with portland cement paint. The effective use of oil and synthetic resin paints represents an independent problem not included within the scope of this recommended practice.

3. The term "portland cement paint" as used in this report, is understood to mean a water dilutable paint in which portland cement is the principal binder constituent. Paint that is manufactured and sold by a paint company and is ready for use except for the addition of and mixing with water is called "commercial paint". Paint that is made up by the user from the basic constituents is called "job-mixed paint". Except where specifically noted otherwise, all discussion and recommendations of this report apply to both commercial and job-mixed paints.

4. Under Recommended Procedure, there are listed the steps necessary to obtain an effective job. Appendixes 1, 2, and 3 discuss various considerations in the composition, manufacture, and durability of portland cement paint and will be valuable for a more complete understanding of what to use and how to use it.

5. The committee wishes to acknowledge especially the valuable contributions of the National Bureau of Standards and the Portland Cement Association in furnishing a large share of the basic data from which this report was prepared.

RECOMMENDED PROCEDURE

Appropriate usage

6. Portland cement paints are suitable for use on exterior and interior portland cement concrete and stucco surfaces except floors or other areas subject to mechanical abrasion. They are especially recommended where it is desired to decorate or reduce the water permeability of exterior walls built of porous, open textured concrete such as concrete block masonry. They also may be used advantageously where the concrete or stucco is damp at the time of painting or may become damp subsequently.

7. Portland cement paints are not well suited for interior surfaces requiring frequent and thorough cleaning, because coatings of these paints are not easily washed and tend to erode with vigorous scrubbing. They are of questionable value for stopping leakage through porous walls exposed to appreciable water pressure, particularly if the paint is applied to the side away from the water pressure. Better results will be obtained if the side exposed to the water is painted. Two or three heavy coats are sometimes successfully employed on the inside surface of the basement walls of houses to stop slight leakage or dampness. Severe conditions usually require more positive waterproofing methods.

8. The resistance of portland cement paints to aggressive solutions and fumes is not significantly different from that of a good quality concrete and because of their thinness they will provide only temporary protection when exposed to severe corrosive conditions. However, they are useful for protecting concrete of poor quality where the exposure is not too severe and it is feasible to repaint as often as may be required.

9. Except where the atmosphere contains reactive fumes, portland cement paint should give satisfactory service under all climatic conditions within the range experienced in the United States.

Age of concrete

10. Best results will be obtained if painting is deferred until the portland cement stucco or concrete, including concrete masonry, has aged at least three weeks. The application of tinted paints to cast-in-

place concrete walls should be postponed longer, several months after the curing period if possible, so that the concrete will be drier and a less likely source of efflorescence. Other advantages of permitting cast-in-place concrete to age longer before applying either white or tinted paints are: (a) weather tends to overcome the effects of form oil; (b) the suction of the surface is more uniform; (c) the paint film crazes less and also tends to cover any checking or crazing which may develop in the concrete surface itself prior to being painted.

Cleaning of surface

11. Surface cleaning prior to painting should consist of thoroughly removing all dirt, dust, form oil and efflorescence. Dirt and dust should be washed off with clean water. Hosing is usually adequate, but scrub brushes should be used if necessary. Soaps or other organic cleaners should be avoided, because unless completely removed they may prevent proper adhesion of the paint. To remove efflorescence first wet the concrete, then apply a 20 percent solution of hydrochloric (muriatic) acid and after about 5 minutes scour off the efflorescence with stiff bristle brushes. If necessary, a stronger solution may be used. It is best to work on small areas, not more than a few square feet at a time. The surface must be thoroughly washed with clean water after the acid treatment. It is practicable to remove traces of form oil with steel brushes, abrasive stones or lye solutions. However, if the surface is generally coated with oil, it will be more effective and economical to sandblast lightly the entire area to be painted or, if time permits, to postpone painting until the oil has been removed by weathering.

12. Some concrete surfaces are so dense (glazed) that adhesion of the paint is affected adversely. Concrete that is cast against plywood, Presd-wood or steel forms frequently presents such a problem. Before painting, such surfaces should be acid washed, lightly sandblasted, or dry rubbed with coarse grit abrasive stones, until the glaze is removed and positive "tooth" and suction for the paint are provided.

13. Old coatings of the organic type must be completely removed before applying portland cement paint. Usually this can be done most effectively and economically by sandblasting.

Wetting of surface

14. Before applying the paint the concrete must be thoroughly wetted to control surface suction and to provide a reserve of moisture to aid in the proper curing (hardening) of the paint. A garden hose adjusted to give a fine spray is well suited for this purpose. A superficial dampening with a brush dipped into a bucket of water does not supply sufficient moisture to be effective.

15. For concrete masonry walls or other concrete that readily absorbs moisture, it is usually sufficient to wet the surface in one operation not

more than one hour before painting. The spray should be applied in such manner that each part is sprayed three or four times for about 5 to 10 seconds each, depending upon the absorptivity of the surface, time being allowed between applications for the moisture to soak into the concrete; the unabsorbed water serves no useful purpose. If the surface tends to dry rapidly, as it may in hot weather, it should be redampened slightly just in advance of painting; it should be moist but not dripping wet or with a noticeable water film when the paint is applied.

16. Dense concrete absorbs moisture so slowly that it should be wetted in at least two operations not less than 30 minutes apart. It will be more effective to dampen rather large areas in advance of painting so that ample time is allowed for the moisture to soak into the concrete. When the paint is applied the concrete surface should be moist but without free water; some suction is essential and will not be provided if the concrete surface is dripping wet.

Preparation of paint

17. Portland cement paints are made ready for use by thoroughly mixing the paint powder and coarse filler, if used, with water in the proper proportions.

18. The proper amount of water will vary some with different products, largely depending upon the fineness of the dry materials. For commercial paints, the manufacturer's instructions should be followed. For job-mixed paints, the proportion of water to powder must be determined by trial. The mixed paint should have the consistency of rich cream except that a slightly thinner consistency is recommended for the first coat applied to open textured types of concrete surfaces such as concrete masonry units.

19. The proportions of water to dry material shown in Table 1 have been found to hold for a variety of paints and are suggested for trial in the absence of definite instructions:

TABLE 1

Paint, dry material, 10 lb	Water, quarts	
	For standard (rich cream) consistency	For medium thin consistency (first coat on open textured concrete)
Commercial paints (constituents mixed and finely reground)	3¼ to 4¼	3¾ to 4¾
Paints, not reground, less than 1 percent retained on No. 100 sieve	2¾ to 3¼	3¼ to 3¾
Paints, not reground, containing coarse filler; more than 25 percent retained on No. 100 sieve	2 to 2½	2½ to 3

20. Mixing procedures vary, but in general it is good practice first to reduce the dry material to a stiff paste with about half the estimated amount of water, after which additional water should be gradually stirred into the paste until the desired consistency is obtained. Preliminary reduction to a stiff paste is necessary in order to break down agglomerates of powder. When the proper proportions have been determined, it is advisable to make all batches the same except when a change in surface texture or other conditions requires a different consistency.

21. The paint should be vigorously stirred for several minutes until color and consistency are uniform and all particles are thoroughly wetted. Workability will be improved by allowing the mixture to soak or prehydrate for 30 to 45 minutes prior to application.

22. The paint should be prepared in batches of appropriate size such that it will be used up within a few hours. Most paints will remain in usable condition for about 4 hours after being prepared; however, in hot weather, some paints, especially those containing calcium chloride, should be used sooner. Keeping the mixed batch in a tightly covered container from which the painters' buckets are filled helps to prolong the usable condition.

23. While being used the paint tends to stiffen slowly due to chemical and physical reactions and evaporation of the water. Thinning the mixture with additional water to reduce consistency is perhaps not best practice, but if done with moderation, it is not considered particularly objectionable for white paints. Retempering of tinted paints should be avoided because of the tendency to alter the color. Vigorous stirring from time to time during the usable period will help to maintain proper fluidity.

Application of paint

24. No painting should be done on frozen concrete or when the paint may be exposed to temperatures below 40 F within 48 hours after application. Some paint manufacturers prefer a minimum temperature of 50 F.

25. When weather conditions are such as to cause the paint to dry rapidly, it is advisable to work "in the shade" in so far as practicable. This makes it easier to keep the surface uniformly moist for proper suction and helps prevent too rapid drying of the paint.

26. To prevent segregation and maintain a uniform mixture, the paint should be stirred frequently in the bucket.

27. The paint should be applied in two coats of the same color. Preferably not less than 24 hours should be allowed between coats; in no case should the second coat be started until the first coat has become sufficiently hard to resist marking by the brush being used. In hot,

dry weather, the first coat should be slightly moistened before applying the second coat. "Doubling back" or the application of a double thick coat is not conducive to good results and should not be permitted. Three-coat work may be employed if desired but the extra coat is seldom worth the additional cost.

28. Portland cement paints cannot be satisfactorily applied with the ordinary hair bristle paint brush. Proper application to coarse textured concrete requires a brush with relatively short, stiff fiber bristles such as fender brushes (commercially sold for cleaning the underside of automobile fenders) or ordinary scrub brushes. This is true even for certain commercial portland cement paints that exhibit smooth working qualities on the order of oil vehicle paints. If the concrete is of smooth or sandy finish, whitewash or Dutch type calcimine brushes are suitable. The paint should be brushed on in uniform thickness and free from "ridging" or excessively heavy brush marks. Laps should be well brushed out.

29. When painting concrete masonry, or other porous concrete for the dual purposes of "moisture-proofing" and decoration, both coats should be vigorously scrubbed on in such manner as to work the paint back into the voids and provide a continuous paint film free from pinholes or other openings through which water might penetrate.

30. Tests of the rain resistance of painted concrete masonry walls* indicate that paints applied with spray equipment provide less protection than scrubbed-on coatings. Spray application therefore is recommended only on dense concrete or interior surfaces or where the paint is not required for waterproofing purposes.

31. While excessively thick films are to be guarded against, there is a tendency in practice to use too much water in the paint and to brush it out too thin. Such coatings may look well at first but generally lose their opacity and protective value much earlier than thicker films. The proper average film thickness and coverage rate is difficult to estimate for portland cement paints because of the effect of differences in the texture of the concrete to be covered. However, for smooth concrete surfaces, a two-coat film should average about 0.015 in. thick; this will be obtained with a coverage rate of about 100 sq ft per gal. of mixed paint for the two coats. If the concrete is rough textured or if the paint contains coarse filler (No. 50 to 100 mesh), a film thickness of 0.025 to 0.035 in. and a coverage rate of from 45 to 55 sq ft per gal. should be sought. Two coats on very rough textured block may require as much as 1 gal. per 35 sq ft of surface. It is questionable economy to stretch the paint much further than indicated by these figures. Thinner

*Copeland, R. E. and Carlson ,C. C.;"Tests of the Resistance to Rain Penetration of Walls Built of Masonry and Concrete", ACI JOURNAL, Nov. 1939, *Proceedings*, V. 36, p. 169.

films could be used, however, when repainting, provided the new and old paints are of the same color and the old paint is in fair mechanical condition.

Curing

32. Proper hardening of paint films of this type depends upon the availability of moisture for chemical reaction with the portland cement. The moisture in the concrete, in the paint itself, and in the air is utilized for this purpose but, except in very humid atmospheres, this is not enough. On most jobs it is practicable to sprinkle the painted surfaces two or three times a day with the same fog spray as used for dampening the concrete. It is recommended that this be done between coats and for at least 2 days following the final coat. The curing should be started just as soon as the paint has hardened sufficiently not to be damaged by the spray, usually about 12 hours after application. Damp curing in this or some equally effective manner must be provided in order to obtain the best results with portland cement paints. It will improve the hardness and durability of the paint in every case and in some instances will mean the difference between a satisfactory and a poor paint job.

APPENDIX 1—DISCUSSION OF COMPOSITION

1. Excluding the effect of procedure, the strength, hardness and density of the paint film are controlled primarily by the portland cement content and secondarily by the inherent durability of the other constituents and their effect on the hardening of the cement-water mixture. In addition, composition influences the appearance or decorative quality of the paint and its application properties.

2. Because of the importance of composition as a factor of paint performance, certain fundamental concepts concerning the functions and value of the various constituents commonly used in portland cement paints are included in this discussion. Available test data and service records suggest desirable percentage limits for most of the constituents, and these also have been included.

Commercial compositions

3. Based on extensive chemical analyses, commercial portland cement paints range in composition about as shown in Table 2.

TABLE 2

Ingredient	Percent by wt.		
	Min.	Max.	Typical white paint
White portland cement	20	91	61
Hydrated lime	0	60	30
Inert fillers (fine silica, limestone, etc.)	0	40	
Hygroscopic salts (calcium or sodium chloride)	0	7	3
Water repellents (stearates)	0	1.5	0.3 (or less)
Opaque pigments (zinc sulfide or titanium dioxide)	0	10	3
Water and organic matter (principally casein)	0	14	2.5 (or less)
Color pigments	0	8	0

4. One or two commercial paints contain coarse inert filler consisting of white silica sand (No. 100 to 50 mesh fraction). Such paints, by virtue of the coarser particles, produce relatively thick films having a characteristic "sandy" texture.

Requirements of federal specifications

5. The Federal Specification for Paint; Cement-Water, Powder, White and Tints (For Interior and Exterior Use), Designation TT-P-21* provides for two types and two classes of portland cement paint with compositional limits as indicated in Table 3 and the following paragraphs 6 and 7.

TABLE 3

Ingredient	Percent by wt.			
	Type 1 Class A		Type 2 Class A	
	Max.	Min.	Max.	Min.
Portland cement	—	65	—	80
Hydrated lime	25	—	10	—
Carbonates (calculated as carbon dioxide CO_2)	3	—	3	—
Water repellents (calcium or aluminum stearate)	1	0.5	1	0.5
Hygroscopic salts (calcium or sodium chloride)	5	3	5	3
Titanium dioxide (TiO_2) Zinc sulfide (ZnS) or mixture	5	3	5	3

6. Class B provides for 20 to 40 percent of white siliceous aggregate (No. 100 to No. 20 sieve), with the balance of the paint conforming to the requirements of Class A. Class B is designed for open-textured masonry.

7. Other provisions and requirements of the specification pertinent to this discussion are:

(a) The paint shall contain no organic binder.

(b) Color pigments shall be limeproof and shall be incorporated by a milling operation.

(c) The total free (unhydrated) calcium oxide (CaO) and magnesium oxide (MgO) in the hydrated lime shall not exceed 8 percent by weight of the hydrated lime.

(d) For dark shades (below 55 percent reflectance) the tinting material may replace the opaque white pigments, titanium dioxide and zinc sulfide.

8. It is to be noted that the typical commercial product conforms closely in composition to the Type 1, Class A paint. The specification indicates no preference between Type 1 and Type 2 except to say that the latter should be used where there is excessive moisture as in water tanks and swimming pools.

Portland cement

9. Tests and experience indicate that the most durable coatings of this type are those in which portland cement comprises a high percentage of the minus No. 100 mesh material. In the Portland Cement Association tests, straight white portland cement films showed negligible erosion and were in excellent condition after 5 years of weather exposure. Replacing 50 percent of the cement by weight with hydrated lime or fine inert filler adversely affected the hardness and weather resistance of the film. Coatings made without portland cement but with lime as the binder were softer and more rapidly eroded than those containing cement. These indications of the importance

*Obtainable from the Superintendent of Documents, Washington 25, D. C., price 5 cents.

of the cement content with regard to paint durability are in accord with experience in the field and the consensus of competent opinion expressed to the committee. Based on these considerations it is recommended that where a paint service life of 5 years or more is desired, portland cement should comprise not less than 65 percent of that portion of the paint powder that passes the No. 100 sieve. For maximum durability or for coatings to be exposed to excessive moisture as in swimming pools, it appears that this value might well be increased to 85 or 90 percent.

Hydrated lime

10. The effect of hydrated lime as a constituent of cement paint may be summarized as follows:

(a) The addition of 10 to 15 percent of lime by weight enhances slightly the whiteness and hiding power of the dry film but may reduce the opacity of the wet film. The effect is not great and depends on the quality and fineness of the lime and cement. A small amount of opaque white pigment is more effective than lime for this purpose.

(b) Workability and particle suspension of the paint mix may be improved depending on the plasticity of the lime used.

(c) In the opinion of some investigators, the use of a small amount of lime seems to increase the miscibility of some water repellents such as calcium stearate, thus shortening the time for proper mixing.

(d) Experimental data on the effect of lime content on the crazing tendency of paint films are very inconclusive except to indicate that the effect is not significant one way or the other.

(e) Depending somewhat on atmospheric conditions there is rather rapid and thorough carbonation of the lime contained in thin coatings and some fairly durable paints containing as much as 50 percent lime have been produced. However, with due consideration to such examples, it seems evident from what is now known about paints of this type that the replacement of an appreciable percentage of the portland cement with lime tends to impair the weather resistance and service life of the paint.

11. It thus appears that although lime is not an essential ingredient it probably may be safely used up to 25 percent of the portland cement content. While it cannot be said definitely that the 25 percent value should never be exceeded, the possible detrimental effects of higher lime content on the durability of exterior films seem to be more obvious than any benefits that might accrue.

Fine fillers (extenders, plasticizers, etc.)

12. The value of finely divided inert fillers such as pulverized silica, limestone, etc., is somewhat obscure. When used in appreciable percentages they have been observed to cause excessive chalking and, consequently, unsatisfactory durability and appearance with continued exposure. Being of about the same fineness as the portland cement they dilute and weaken the cement matrix. Even though such fillers may reduce shrinkage somewhat or enhance slightly such properties as brushability, it appears that their overall effect is more likely to be detrimental than beneficial.

13. Small amounts of colloidal material such as plastic clay, etc. are sometimes added to improve the workability and suspension qualities of the paint mixture. Their effect on the durability of thin cement coatings is not very well known. Probably they should be avoided or at least restricted to small amounts (2 or 3 percent) until more thoroughly tested as a paint ingredient.

14. The use of organic compounds for better dispersion and workability is not recommended except where they have been thoroughly investigated and shown not to be detrimental to the paint film.

Coarse fillers (aggregates)

15. Favorable results have been obtained with relatively coarse fillers, usually a siliceous material of acceptable color and weathering qualities such as fine white silica sand. In general the softer sands such as those derived from limestone and marble have proved less satisfactory as paint aggregate. The sand should pass the No. 20. sieve with not less than 90 percent retained on the No. 100 sieve; the No. 100 to No. 50 mesh fraction is perhaps the most satisfactory for this purpose. Because it is much coarser than the cement, such filler does not weaken the cement matrix as do the very fine fillers. The sand may be used in varying amounts but should not exceed the proportion of portland cement. Based on experiences on several jobs and on 2 to 4 year exposures of painted concrete masonry specimens included in a Portland Cement Association investigation, such filler appears to improve the surface hardness and durability of the paint. The sandy texture of the paint is considered attractive by many persons and it insures excellent adhesion between successive coats.

16. Mixtures containing sand cannot be applied with a hair brush but readily go on either smooth or rough textured concrete if a stiff bristle brush is used. The use of aggregate introduces a minor inconvenience in requiring more frequent stirring of the mixture in the paint bucket in order to keep the sand in suspension.

17. It is not recommended that aggregate be made a required constituent of portland cement paints, but its indicated value in paints that are to be applied to concrete masonry or other coarse textured concrete should not be overlooked.

Hygroscopic salts

18. Hygroscopic salts are added to portland cement paints as curing aids. Presumably they help keep the cement moist for several days with water drawn from the air, but their efficacy in this respect has not been definitely established for all conditions.

19. Calcium chloride accelerates the rate of hardening and in amounts up to 4 or 5 percent has no significant effect on ultimate strength. At this concentration, the salt apparently is not harmful as to the shrinkage, weather resistance, opacity, or other properties of the paint film. It is a possible source of efflorescence, but no evidence has been presented indicating that it is of a serious nature. Pending the development of additional data, its use in amounts not exceeding 5 percent is recommended.

20. Sodium chloride (common salt) is another salt that is sometimes used. However, it is not nearly as hygroscopic as calcium chloride and there is evidence that it tends to reduce ultimate strength. Calcium chloride is preferred also because of its definite accelerating action, which is desirable for reducing the curing period and the time during which the paint, in an unhardened state, might be damaged by rain or freezing weather.

Water repellents

21. Many of the commercial paints contain water repellents, usually either aluminum or calcium stearate powder ground with the base. Sometimes paste forms of stearate are stirred into the mixing water and added on the job. If powdered stearate is added on the job it must be very thoroughly mixed with the white portland cement and other dry ingredients prior to adding the mixing water; in such cases it is preferable and more convenient to use a waterproofed cement in which the stearate has been intimately mixed by a grinding operation.

22. The comparative merits and optimum amounts of the different water repellents as regards some properties of portland cement paint, particularly durability and shrinkage, are not well known. Some tests have been made on paints containing stearates incorporated both by grinding and by hand mixing and in addition there are some data on the effect of these substances on the strength and absorption of portland cement

mortars. These data indicate that the amount of pure stearate in the paint should range between 0.5 to 1.0 percent of the portland cement content if water repellency and low absorption are to be obtained without too great a reduction in strength. However, some authorities contend that 0.3 to 0.4 percent of stearate is sufficient when it is ground with the paint and therefore is more homogeneously incorporated than is possible by hand mixing. Tests have shown that 2 percent of calcium stearate ground with the base seriously reduces the strength and hardness of the paint film but that 1 percent can be safely used. It also has been tentatively indicated that 1 percent of calcium stearate appreciably reduced the amount of crazing, but 0.5 percent or less was ineffective in this respect. Water repellents tend to add somewhat to the workability of the paint and the opacity of the wet film. Their effect on weather resistance has not been specifically determined as yet but it should be favorable. Proper damp curing and pre-wetting of the receiving surface are especially important when applying paints which contain water repellents because once dried they are difficult to re-wet.

23. It thus appears that small amounts of water repellents are not detrimental to the paint and offer certain advantages which may well justify their use. Not more than 1.0 percent by weight of the portland cement content is recommended.

Opaque pigments

24. White paint should contain a small amount of opaque white pigment, such as titanium dioxide or zinc sulfide, for the purpose of increasing the whiteness and hiding power of the film. Amounts of the commercially pure pigment of from 2 to 5 percent by weight of the portland cement content are sufficient and can safely be used. Due to their fineness it is possible that appreciably larger amounts might be detrimental as to shrinkage, crazing, and strength. Opaque white pigments should also be used in light tints such as cream and ivory, but are unnecessary in the darker colored paints.

Color pigments

25. Color pigments should be commercially pure oxides, very finely ground and lime proof. Different commercial grades vary in quality and price. Only the best pigments, with high tinting strength, should be used in paint. The amount of pigment required to give the color value desired depends upon its tinting strength and dispersion in the paint. For best results tinted paints should be produced in a grinding mill.

Organic substances (casein)

26. Casein or other organic binder is contained in some portland cement paints presumably to improve workability and the adhesion of the paint to nonporous surfaces. Even small amounts of such material may prevent the proper hardening of the cement and thus seriously impair the durability of the coating. Their use in portland cement paints, particularly those intended for use on exterior concrete or stucco surfaces, is therefore not recommended.

Water

27. Any clear water from a domestic source is suitable. The amount of water required to reduce the dry material to painting consistency is discussed in paragraphs 17 to 20 inclusive of "Recommended Procedure."

APPENDIX 2—MANUFACTURE AND STORAGE

1. For best results, the paint powder throughout all batches used on any one job must be uniform in composition and quality. The ingredients, therefore, must be accurately proportioned by weight, and thoroughly mixed together.

Commercial paint

2. In the manufacture of commercial paints, the mixture of finely divided ingredients exposures of painted concrete masonry specimens included in a Portland Cement generally is reground in a suitable ball mill or pebble mill. This results in better dispersion of the ingredients and thus in a more uniform product than can be obtained with ordinary job mixing methods. The finer grinding also enhances the suspension and brushing qualities of the paint.

3. Plant manufacture permits the milling in of tinting pigments with the base, which is essential for perfect uniformity of color. This is the basis for the recommendation that only commercial paints be used where colors other than white are desired. Powdered water repellents, aluminum or calcium stearate, also should preferably be milled in with the base.

4. While satisfactory results have been and are being obtained with paints prepared on the job, the convenience and advantages of using a properly-formulated commercial paint are obvious.

Job-mixed paint

5. Where a white paint is proposed, economy may sometimes be realized through the use of job-mixed paint.

6. Appendix 1 gives detailed data on ingredient proportions. It is recommended that these data be studied before determining the mix to be used. Following is a typical mix which will be found effective under normal conditions where the coating has the primary functions of decoration and water repellency:

White portland cement	75 percent
Hydrated lime	17 percent
Opaque pigment (zinc sulfide or titanium dioxide)	3 percent
Calcium chloride	4 percent
Calcium or aluminum stearate	1 percent

7. Where sand is to be incorporated with the paint, it should be held to not more than 50 percent of the total. The sand should be dry mixed with the paint powder before the gaging water is added.

8. The minimum mixing considered essential to satisfactory results consists of thoroughly stirring the dry ingredients together in a suitable container, and then sifting this mixture through a No. 14 mesh fly screen. Batches of 100 lb or more may be mixed in a small stucco or mortar mixer.

9. Paste water repellents and calcium or sodium chloride, if used, should be mixed with the gaging water.

Storage

10. Because of the hygroscopic properties of portland cement, the paint powder must be protected from the atmosphere while in storage. Particular care must be observed to maintain containers airtight where hygroscopic salts have been included in the paint.

APPENDIX 3—GENERAL CHARACTERISTICS AND FACTORS AFFECTING DURABILITY

1. Portland cement paint was developed specifically for application to concrete and masonry surfaces. Since portland cement and water are the principal constituents, the resulting paint film may be considered in much the same light as any thin layer of hardened portland cement.

Resistance to moisture and alkali

2. One of the more important qualities of portland cement paint is its tolerance to alkaline conditions to which coatings on concrete are frequently subjected. Since the film and the substrate are basically the same, the alkali from the concrete that is harmful to most organic paint films has no damaging effect.

3. Prolonged saturation, alternate wetting and drying, and freezing and thawing constitute conditions harmful to any paint film, but are withstood by coatings of the portland cement type probably as well as by other types.

Appearance

4. The typical film possesses good decorative qualities as to hiding power and color. When wetted, as by rain, it becomes slightly translucent or darker in shade and appears somewhat less satisfactory than when dry. However, on drying out, the film returns to its original color and opaqueness. Tinted paint films also assume a deeper shade of color when wetted.

5. Poor opacity and color value may be due to some deficiency in the paint composition, to the use of too much water in the paint mix, to applying the paint too thin, or to the continued presence of excessive moisture behind the film.

6. The appearance of tinted paints is marred if efflorescent salts are deposited on the film surface. Much of the efflorescence trouble is caused from painting concrete at too early an age while it is still emitting appreciable excess moisture. Especially with tinted paints, it is good practice to postpone painting as long as feasible. They should not be applied to concrete that is in direct contact with water containing sulfates or other salts which might permeate to the painted surface. Frequently it is feasible to build the structure so that extraneous water will be excluded from the areas to be painted.

7. All paints suffer in appearance from normal accumulation of dirt and soot stains. Opinions differ as to whether portland cement paints are especially susceptible to soiling and staining due to their greater porosity as compared to oil and resin bound films. This matter has been under observation for some time in paint studies being conducted by the Portland Cement Association. In approximately 500 specimens exposed from 18 months to 5 years to rather bad sooting conditions the organic paints, on the average, became soiled at about the same rate as the portland cement paints.

Service life

8. By service life is meant the time required for the paint to depreciate physically to a point at which it no longer provides adequate protection or coverage. According to this concept it is not determined by a reduction in decorative value, providing that the poor appearance does not result from physical deterioration and can be remedied by cleaning rather than by repainting.

9. Based on information from several sources it appears that well-formulated portland cement paints properly applied to exterior wall surfaces exposed to the weather may be expected to have a service life of about 8 years. Some paint jobs last considerably longer, some not as long. The less durable jobs frequently are those on smooth, dense concrete surfaces where there is the tendency to brush out the paint too thin and to neglect the requirements for satisfactory adhesion.

10. It is logical that paints on interior surfaces or other areas protected from the weather should last longer than when applied to exterior surfaces. Frequent cleaning however may markedly erode the film and shorten its life.

Adhesion (peeling, blistering)

11. Portland cement paint films generally adhere satisfactorily to the concrete and between coats and seldom fail by peeling or blistering. Where these defects do occur,

they almost invariably are due to inadequate preparation of the surface. Portland cement paint will not adhere properly to surfaces that are glassy smooth or are soiled or coated with foreign matter. Such surfaces must be well cleaned and, if necessary, acid washed or sand blasted to open the pores and provide suction and "tooth" for the paint. A weak bond results if the concrete is not dampened prior to painting and the paint film is allowed to dry rapidly. It appears that adhesion is influenced more by differences in painting procedure than by composition although there is some evidence that flaking may be caused if the paint contains an excessive amount of unhydrated lime or water repellent.

12. Since very few cases where portland cement paints have failed by blistering are on record, little has been learned as to its cause. The presence of excess or free water on the surface being painted and the use of too much water repellent in the paint have been suggested as possible causes, but these have not been definitely verified.

Crazing

13. The typical portland cement paint film is hard, strong and relatively brittle. This latter property together with drying shrinkage sometimes results in fine crazing or checking. Exterior films seem to be more susceptible to this trouble than films which are protected from the weather.

14. Other characteristics of the film that influence crazing are tensile strength and extensibility, which in turn vary with the composition of the paint. However, the observed differences in crazing due to the ordinary variations in composition are not great.

15. Normal crazing is not progressive, generally being completed within a few weeks after starting and in itself is not a serious defect. Observations by the Portland Cement Association on several hundred test specimens over a period of 4 years showed no significant change in the extent of crazing after the first year.

16. Serious crazing, involving fractures extending through the paint film and more than 0.002 in. wide, usually is related to and induced by the development of crazing of the concrete surface subsequent to painting. Crazing over mortar joints also is usually traceable to fine shrinkage cracks in the mortar.

Chalking

17. Rapid chalking is a characteristic quality of soft, weak films and leads to proportionately rapid deterioration in the color, opacity and durability of the coating. Usually it is due either to a deficiency of portland cement in the paint or to the failure to provide adequate curing, which is necessary to the proper hydration and hardening of the cement. Soft films also may be caused by ingredients such as casein or other organic compounds, which prevent normal reaction between the cement and water.

Pinholes

18. A few pinholes in the film are not objectionable unless the paint is depended upon to reduce effectively the permeability of the surface. They result when the paint is not worked into the surface voids or when entrapped air breaks through the fresh film where it bridges instead of fills the voids. Pinholes can be eliminated by vigorous brushing. Composition is a slight factor in causing this defect only insofar as it affects the workability and surface tension of the paint. Sprayed coatings usually contain more pinholes than brush applied films, especially when the receiving surface is coarse textured or contains many deep pits or voids.

Fastening heavy machinery to concrete bases and walls often is a problem, especially when great accuracy in placement is required. As a bearing and grouting material dry mortar was the answer in several unusual cases discussed by Commander Brooks.

Dry Mortar as a Bearing and Grouting Material*

By BOYD S. BROOKS†

SYNOPSIS

In 1939, at the navy's new David Taylor Model Basin at Carderock, Md., there was an urgent need for a non-shrinking bearing material to be used under the cast-steel "chairs" which support the rails on which the towing carriages run. Requests for information concerning materials and installation procedures produced such varied answers that investigation was started to find the best material and the best procedures for placing it.

The bearing material selected for use was 1:3 portland cement mortar, with 4.13 gallons of water per sack of cement. This was rammed firmly into place.

This bearing material has now been in place over a period of some 9 years under extremely severe conditions and without exception has proved to be excellent in function and condition.

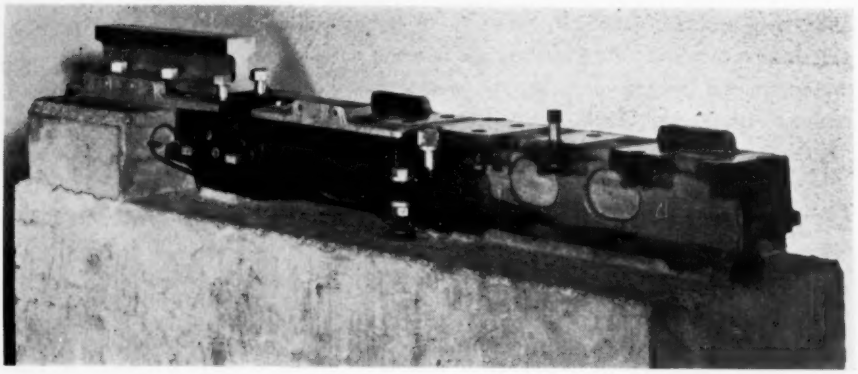
INTRODUCTION

One of the purposes of the David Taylor Model Basin is to predict the behavior of ships by the testing of scale models. Model testing demands continuous precision work because inaccuracies are magnified in transferring model data to full scale design. Some of the tests require that the models be towed through the water by a carriage which straddles the towing tank and runs on rails securely anchored to the concrete walls of the tank (Fig. 1). It was therefore most necessary to have an excellent bearing material to fill the gap between the supports for the towing carriage rails and the top of the concrete walls. This gap varied in size from ¾ in. to 2½ in. The principal requirements were that the filling material have a reasonably high compressive strength, that it should not expand or shrink after placing, that it conform closely

*Received by the Institute Aug. 25, 1948. Title No. 45-19 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 5, Jan. 1949, *Proceedings* V. 45. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than May 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

The opinions and ideas expressed are those of the author and not those of the Naval Service.

†Member American Concrete Institute, Commander C.E.C., U.S.N., Executive Officer, U.S. Naval School (C.E.C. Officers), Naval Construction Battalion Center, Naval Station, Port Hueneme, Calif.



Official photograph—U. S. Navy

Fig. 1—Cross-section model of rail and supporting structure. The 1:3 packed mortar has been placed beneath the cutaway section of the chair. The view of the model shows the basin wall, the cast-steel chairs, the chair tie-down bolts, the mortar beneath the chairs, the ballast concrete about the chairs, and the finished rails.

to the under side of the steel supports and the top of the concrete walls, that it be easily placed by laborers on a production scale, and that it be reasonable in cost. The rail tops had to be maintained on an earth's circumference plus or minus 0.0015 in.

The mix and moisture content of the mortar was decided upon after volume change tests were made by the National Bureau of Standards on various mixes (some of which contained admixtures) having different moisture contents. Capt. Hugo C. Fischer of the Civil Engineer Corps of the U. S. Navy, his staff, and the late John J. Earley of Rosslyn, Virginia, made valuable suggestions and comments.

A relatively dry cement-sand mortar was selected as the bearing material under the supports for the towing-carriage rails. It was also found to be valuable for use under machine-tool bases, and for fastening bolts into holes in concrete. The grouting was started in 1939. In January 1948, a report* was published giving in detail, with many pictures, the procedures followed for the conduct of this work.

DESCRIPTION OF MORTAR

A clean, well-graded Potomac River concrete sand whose fineness modulus varied from 2.44 to 2.66 was used with a normal portland cement.

The mortar was proportioned 1 part of cement to 3 parts of sand by weight with water content adjusted so that a mass of mortar squeezed in the hand would retain its shape (Fig. 2). The consistency was that of foundry sand or of damp cornmeal and was such that when the

*"The Use of Dry Mortar as a Bearing and Grouting Material at the David Taylor Model Basin" by Comdr. B. S. Brooks, C.E.C., U.S.N., Jan. 1948, Report 538, the David W. Taylor Model Basin, U.S.N., Wash., D. C.

Official photograph—U. S. Navy

Fig. 2—The dry mortar has a consistency of damp corn meal and it definitely looks rough and dry. The cement and sand are first mixed dry, water added, thoroughly mixed with a trowel, and then the material mixed by rubbing and rolling between the gloved hands.

mortar was smoothed with a trowel, four or five strokes would cause a slight appearance of water on the surface. With the cement and sand used, 4.13 gallons of water per 94-lb bag of cement produced the described consistency.

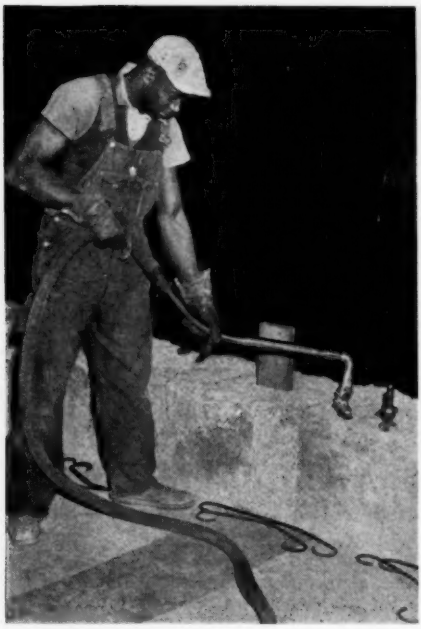
Batches of mortar consisting of 7 lb cement with sand and water in proportion were hand mixed, after preliminary dry mixing, by rubbing and rolling between rubber-gloved hands. No remixing was done and no setting period was thought necessary.

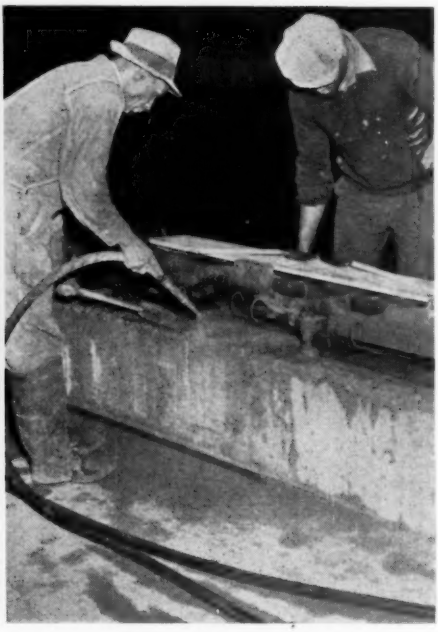
RECOMMENDED GROUTING PROCEDURE FOR A BEARING MATERIAL

Roughening the concrete surface.

A concrete surface must be well roughened before grouting to increase the bond between the old concrete and the new material.

Chipping, the first step in cleaning the old concrete, removes the top layer of concrete that is usually weaker than the rest and removes the material that may be oil soaked, painted, or otherwise covered with foreign material. It is most important that all dust from the chipping operation be removed, as concrete dust has no cementing value. Dust is best removed dry, preferably by suction. In the work described an industrial vacuum cleaner (Fig. 3) was used for the rails but for other work such as grouted-in bolts, a blower was used.





Official photograph—U. S. Navy

Fig. 3 (left)—An industrial vacuum cleaner was used to remove dust from the chipped concrete surface. A vacuum cleaner has advantages over the blower because the fine material is removed from the surface pores and the dust is prevented from flying into the air and settling on clean surfaces elsewhere.

Fig. 4 (right)—The wall was thoroughly scrubbed with water and stiff fiber brushes. This operation was the beginning of the concrete wetting period which continued for several days.

The concrete surface should then be scrubbed (Fig. 4) until the water is clear in the irregularities of the horizontal surfaces and in the bottom of drilled holes. A fiber brush should be used for scrubbing, as a wire brush will continue to scour the surface and the water will never become completely clear. Wet sand blasting gives a satisfactory surface but spreads through the room not only the removed concrete, but the blasting sand as well.

Wetting old concrete surface

After roughening, the surface should be thoroughly wetted, for several days if possible. This can be done by placing soaked burlap over the horizontal surfaces, by keeping vertical holes filled with water, and by forcing water into horizontal holes after they have been stuffed with rags.

There are two distinct reasons for thoroughly wetting the old concrete:

1. Wetting prevents the old concrete from drawing the mixing water from the new mortar, where it is needed to hydrate the cement.

2. Wetting reduces the differential stresses which tend to form over the area of contact between the old and new material. These shrinkage stresses are due to uneven drying out of the old and new materials; the new material and the first few inches of the old concrete should have comparable moisture contents and should dry out together.

Removal of free water from surface of the old concrete

Before the bonding grout is placed all *free water* should be removed from the old surface (Fig. 5). Surface dry concrete is dull gray in appearance; a concrete surface with free water will glisten.

Excessive water on the surface of the old concrete will create a layer of weaker mortar of a thickness depending on the quantity of excess water. This excess water will reduce the bond between the two surfaces as the pores of the old concrete are filled with water, thereby eliminating any suction on the bonding grout by the concrete and reducing the amount of grout that can be forced into the pores. The practice of absorbing excess water by brushing on dry cement is one method of placing bonding grout. This is not recommended, though it is far better than no bonding grout at all.

Bonding grout application

The last step in the preparation of the old concrete surface is that of scrubbing on the bonding grout (Fig. 6). This grout is a mixture of cement and water, usually with a consistency of cream but stiffer mixtures may be desirable on vertical walls and in bolt holes. It is

Fig. 5—The wall was kept wet for at least 2 days before the placement of mortar under the chairs. A dry concrete surface permits definite suction by the concrete of the fluid bonding grout. This ideal was seldom obtained and it was necessary to remove the surface water by the use of an electric hand blower.

Official photograph—U. S. Navy

Fig. 6—A bonding grout of cement and water, having a consistency of pea soup, was scrubbed onto the concrete surface after the concrete was surface dry. Special 3-prong bristle brushes were used.

Official photograph—U. S. Navy

believed that bonding grout made of cement and water is better than that made of cement, sand and water, as sand is only an inert filler that adds nothing to the real purpose of the grout. It is most important that the grout not be placed so far ahead as to become dry before the new mortar is placed, as incomplete hydration of the cement in this thin bonding grout will result. The grout should be scrubbed on in a thin layer, as though one were trying to scrub it off. A fiber brush is usually used, and bottle brushes are useful in drilled holes.

Packing mortar in place

Dry mortar can best be packed under a machine or a base plate by backing up one side of the open space and placing all the material in from the other side. The material can be placed and rammed by rectangular and wedge-shaped paddles of the size, shape, and length demanded by the particular job (Fig. 7). When the space beneath the plate has been filled, the backup block is removed and the face of the mortar is rammed from the opposite side. Mortar around bolts can

Official photograph—U. S. Navy

Fig. 7—Heavy paper was used to keep the previously cleaned concrete surface from being covered by the mortar, except the area where the mortar was to be placed immediately under the chair. All material was placed from the walkway side because of the larger working area. Rammers of various sizes and shapes were used as shown; note the metal strip lying on the board above the chair.

best be packed by using a pipe of the size that will fit over the bolt and into the drilled hole. A hammer may be used to advantage on the end of the pipe.

Thorough packing does several things:

1. It forces all particles within the mass into more intimate contact, thus assuring a strong, solid filling and assuring the hydration of the cement.
2. It forces the dry mortar to assume the contours of the under side of the structure immediately above and of the roughened concrete below.
3. It effectively eliminates shrinkage as the mortar is placed under some initial compression.

Curing the newly placed mortar

It is necessary to retain the moisture that was originally placed in the mortar. Of course, the first step is to make sure that the old concrete will not draw out the moisture from the new mortar; the

second step is keeping the moisture in by covering the surface with wet burlap and keeping it wet for periods ranging from two days to three weeks, the length of time depending on the humidity and temperature. For the work described in this report the usual time was three to five days, under cover of wet burlap.

Burlap worked with the hands while immersed in water is preferable to burlap merely sprinkled with water. Curing should be started as soon as the surface can support the weight of wet burlap and when the new material is hard enough to prevent the curing water from leaching the cement from the new mortar. It is most desirable to keep the direct rays of the sun from newly placed cement and mortar.

RESULTS

One might logically ask what can be expected of a mortar placed under the "chairs", the machine-tool bases and column bases by following the above procedure. Judging from the work that has been done at the Taylor Model Basin over a number of years, using laborers familiar with the process, the following results were achieved:

1. Intimate, perfectly smooth contact was secured with 98 percent of the area of the under side of the metal base. The contact surface has the smoothness of glass.

2. Bond with the old concrete base of at least 95 percent as evidenced by not over 5 percent cleavage along the surface of contact upon removal of the new mortar.

3. Compressive strength at 28 days of representative specimens (¾ x 3 x 12 in.) of 10,000 to 12,000 psi was secured regularly.

4. Excellent results in all mortar thicknesses; those used varied from ¼ in to 3 in.

5. Compared with sloppy grout, the cost of the method outlined here was less, the working space required was smaller, and the mess and trouble were far less.

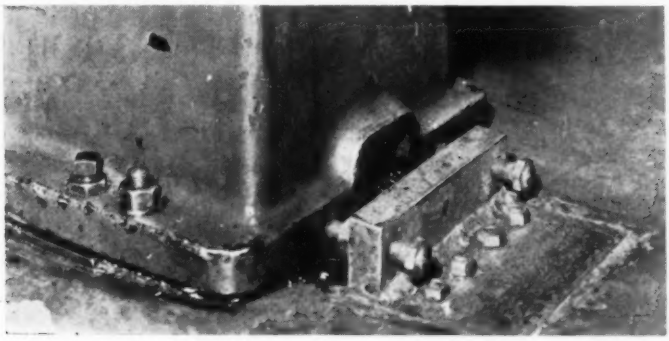
6. No trouble was ever experienced with the machines or bases moving upward due to the pressure exerted by packing the mortar in place.

7. Some of this mortar has been in place for 9 years. Close observation of the many bases, some under machines having exceptionally severe vibrations has shown no disintegration of the mortar bedding and no shrinkage of it away from the metal base. Whether to remove wedges or to leave them in place appears to be immaterial, except for appearances.

GROUTING BOLTS IN CONCRETE

This dry mortar is not restricted to use solely as a bearing material* under machine tools and base plates (Fig. 1, 8). Probably its most frequent use is for grouting bolts in holes drilled in concrete. Where

*Also described by the author in the complete navy report, (see note, p. 370) are: use of dry mortar to repair temperature cracks; dry mortar for building scale model of river bottoms; floor patching with dry mortar; dry mortar as load transfer medium between overloaded concrete floor and steel I-beams.



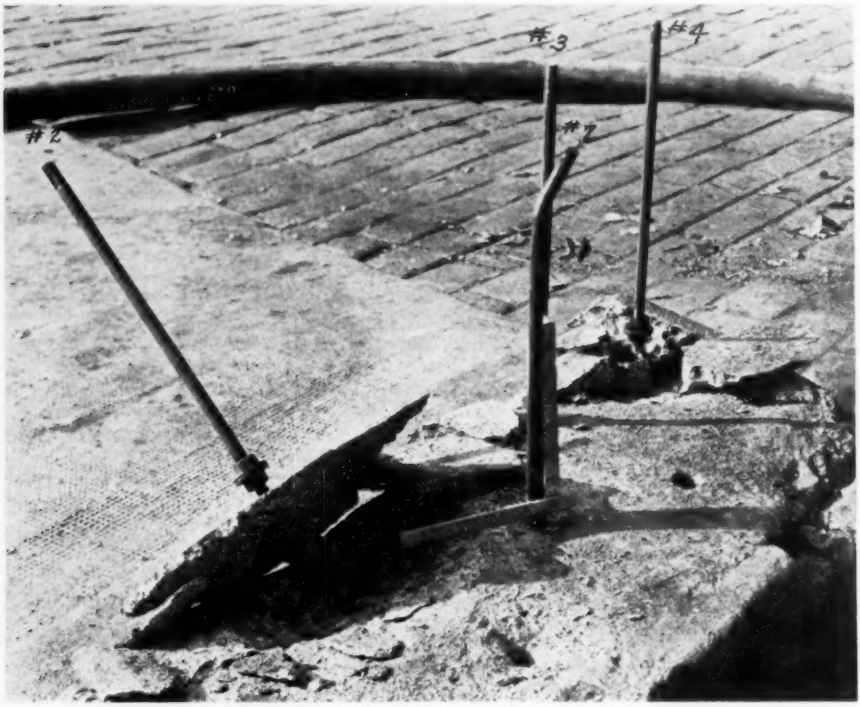
Official photograph—U. S. Navy

Fig. 8—Weight of the high speed planer was not sufficient to hold it in place under the severe vibration. A heavy bolster plate placed in the floor as shown has proved satisfactory. The floor was recessed to receive the plate, four bolts grouted in, the plate placed on a mortar bedding and tightened down. The locking screws were adjusted as required.

precision of placing bolts is a major consideration, it is believed that the cheapest and most satisfactory way of installing ordinary holding bolts in concrete is by drilling holes and grouting in the bolts with dry mortar. The two wind tunnels at the Taylor Model Basin rest on 28 separate pedestals and are held down by 216 bolts. Because it was expected that the pedestals would be poured at different times and the bolts were to be placed within a tolerance of less than $\frac{1}{8}$ in., the specifications called for the bolts to be placed by grouting them into drilled holes. After all of the pedestals had been poured, the bolt locations were determined, the holes were drilled and the bolts grouted. Their placement was well within the tolerance required and the method of placing quite economical.

Literature giving detailed information on grouting bolts is practically unknown. In 1939 the U. S. Experimental Model Basin, Navy Yard, Washington, D. C., published a short report by the writer, in which a comparison was made between grouting of $\frac{1}{2}$-in. bolts in concrete using this relatively dry mortar and placing them by using a patented lead-expansion washer. The mortar method was found far superior as the failures under direct pull occurred by disruption of the concrete when bolts were grouted in while the lead washers permitted continuous slip under a maximum load which was only a fraction of what the grouted-in bolts withstood (Fig. 9). Using $\frac{1}{2}$-in. bolts in 1-in. diameter drilled holes, 3 in. deep, no slip could be measured before final failure of the concrete.

In general, the various steps in the process of grouting a bolt in a hole drilled in concrete (Fig. 10) are similar to those described for grouting beneath plates. A few additional details must be noted.



Official photograph—U. S. Navy

Fig. 9—The ½-in. rods were grouted into an old concrete wall. Holes were 3 in. deep, 1 in. in diameter. Standard 1:3 mortar used and bolts pulled on the 15th day. All bolts withstood a 10,000 lb. pull for a short period. The open hole at the right was used to place a bolt using an expansion washer. The hole in center foreground had approximately a ½-inch of mortar above the expansion washer, which caused the surface of the concrete to be pulled off. Bolts were pulled using dynamometer and screw jacks as beam supports. Bolt No. 7 was not pulled to failure as it was originally tested before the apparatus was arranged to give a direct vertical pull.

Locating and drilling holes

The bolt location, may be marked with a large cross, so that when the hole is drilled the exact bolt location will not be lost. In the work described in this paper, the diameter of the drilled hole was made ¼ in. larger than the greatest distance across the corners of the bolt head. If a nut is used at the base it must be tight and spot-welded to the bolt. No attempt was made to determine what depth of hole was required to develop the full strength of the bolt. The minimum depth for a ½-in. bolt has been taken as 3 in., the maximum being the distance to the reinforcing steel.

Care must be exercised in drilling, as it is far easier to check the plumbness of a hole as it is drilled than to reshape the hole afterwards.

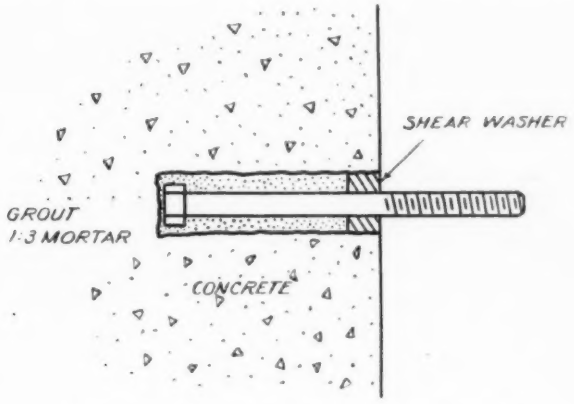


Fig. 10—A grouted-in bolt in existing concrete

Cleaning and grouting

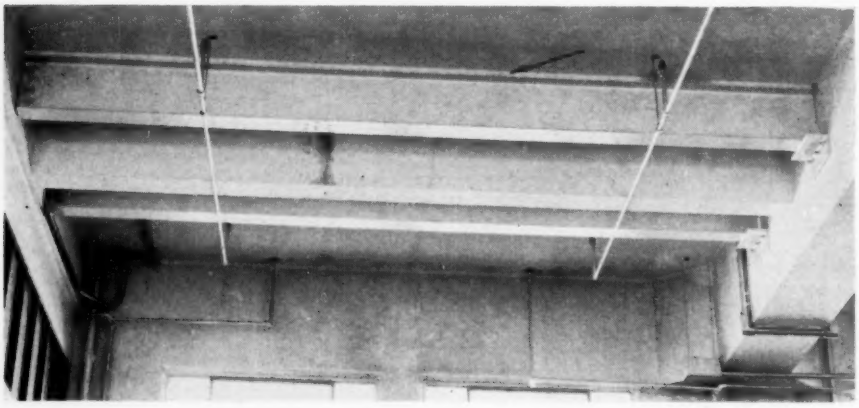
After the hole is drilled, it is cleaned while dry, then cleaned with water. Water is left in the hole for a couple of days, then removed and the side of the hole is surface dried. Bonding grout is scrubbed on, and here a fiber bottle brush is found useful. A bit of bonding grout can be placed in the bottom of the hole to assure that there is material around the sides of the bolt head so as to eliminate the possibility of the bolt being able to turn after being grouted in. The standard 1:3 grout, with 4.13 gallons of water per sack of cement, can be placed around the bolt head by a small ¼-in. rod or by a pipe of such size to fit within the hole and around the bolt. Effective packing can be obtained by this pipe method, especially if additional force is furnished by hammering. The mortar around the bolt is cured by covering with wet rags for several days.

Shear washers

Bolts which must resist forces normal to the bolt length may be provided with "shear washers," a steel washer having an outside diameter to fit into the drilled hole and with an inside diameter to fit snugly over the bolt (Fig. 10). This increases the bearing area of steel against the concrete. One may place, in the space left for the shear washer above the dry-packed mortar, a small quantity of bonding grout to permit it to be forced out past the washer and fill any space remaining. The depth of the shear washer has been arbitrarily taken as ¼ in. plus the depth of the bolt head, the ¼ in. being the depth of concrete which may have spalled or cracked at the surface due to drilling.

Service record

Approximately 1000 bolts of different sizes and lengths have been grouted in concrete slabs, concrete structures and solid rock at the



Official photograph—U. S. Navy

Fig. 11—Installation of I-beams on the underside of an overloaded concrete floor slab to transfer the extra load to underloaded concrete girders. I-beams were placed on either side of a concrete beam. Note that the I-beam is supported on the right by an angle bracket and on the left by clip angles; this was necessary due to the location of the tensile steel.

station over the several years, and they have definitely proved that the method of grouting bolts in drilled holes with this relatively dry mortar is satisfactory, economical* and precise.

AS A LOAD TRANSFER MATERIAL

Dry mortar has been used with great success as a load transfer material for both machinery and structures. An outstanding example is shown in Fig. 11 where an overloaded concrete floor slab was reinforced with steel I-beams to transfer the load to underloaded concrete girders.

All bolts were grouted into the concrete. Before placing the mortar between the top of the I-beams and the underside of the concrete floor slab, wedges were placed deflecting the floor upward and the beams downward. These wedges were removed after placing the mortar and the resulting holes filled with mortar. In this way it was assured the I-beam took the overload from the slab above.

*The author has had considerable experience with both types of base grouting. In one instance bases of vertical cantilever beams were grouted to the concrete floor before being tested under impact loads. Sloppy grout was used. When the bases were later removed areas estimated at 30 percent of the grout area had no contact whatever because the space had been filled with air and water. It had taken two young graduate engineers about two hours to put the sloppy grout under the base using stiffer material around the sides and irregular-shaped blocking-in forms. It is estimated two laborers could do the same job with dry mortar in less than an hour, with better results.

Mr. Bakhoum discusses a time saving technique in designing rectangular sections by use of bending, compression and tension curves to obtain section dimensions directly rather than by assumption of a reasonable section.

Direct Dimensioning of Rectangular Sections*

By MICHEL BAKHOUM†

SYNOPSIS

On the basis of the standard theory for reinforced concrete design, methods have been derived for the direct dimensioning of rectangular sections, with or without compression reinforcement, when subjected to simple bending, eccentric compression or eccentric tension. Two cases are considered: when the depth of the section is fixed and when it is not fixed. The conditions leading to minimum reinforcement are also studied. The solutions developed have been simplified by the use of curves which are applicable for pure bending as well as for eccentric compressive or tensile forces and cover any values of f_c, f_s and n. The curves allow making a variety of designs in a relatively short time and thus help in choosing the most economical or most suitable section.

INTRODUCTION

This paper is intended for the use of the practical designer in his everyday designs of rectangular sections subjected to pure or compound bending with eccentricities beyond the kern of the section. The case of small eccentricities within the kern is very simple and is not dealt with here. The paper is based on the general assumptions of the standard theory for reinforced concrete design.

Some difficulty is encountered in designing rectangular sections, with tension and compression reinforcement, under eccentric compression or eccentric tension. In many cases the designer assumes a reasonable section and checks whether the fiber stresses have not been exceeded. This procedure, besides leading to a waste of time does not allow a flexibility in design, as the designer is generally content with the first section that leads to safe fiber stresses. It is well known that for any given force or couple there is not a unique section that is required to resist it. A method of design that gives some alternative solutions in

*Received by the Institute April 19, 1948. Title 45-20 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 5, Jan. 1949, *Proceedings* V. 45. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than May 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

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a reasonably short time will certainly lead to the choice of a more suitable or more economical section.

Also, it is general to treat the cases of simple bending, eccentric compression and eccentric tension separately while they all can be dealt with together.

In this paper, methods have been derived for the direct dimensioning of rectangular sections without the need for checking the stresses after that. The methods have been simplified by the use of curves which are unitless, applicable for simple bending, eccentric compression and eccentric tension, and cover any values of f_c, f_s and n within the practical range.

In a rectangular section, the unknowns are the width b, the depth d, the tensile reinforcement A_s and the compression reinforcement A'_s. It will be always assumed here that b is a given value. It is clear that, since for normal stresses we have only two equations of equilibrium, there is no way of getting the remaining three unknowns except if an extra assumption is made—such as fixing the depth or the ratio of the compressive to tensile steel. Both cases, when the depth is fixed and when it is not fixed, are dealt with here. A study is also made for the conditions leading to minimum reinforcement in the first case.

The methods given here are general and are not made according to any particular specifications. However, at the end of the paper they are adapted to the recommendations of the 1940 Joint Committee.*

A summary at the end of this paper gives a short-cut for the whole procedure. It may be helpful to read the summary first.

This paper has been condensed from one prepared in 1938 under the supervision of Prof. W. S. Hanna, head of the structural engineering department at Fouad I University, Giza, Egypt, to whom the writer is greatly indebted.

DESIGN WHEN THE WIDTH AND DEPTH OF THE SECTION ARE FIXED

The analysis made here is for the case in which b and d are fixed values and it is required to determine A_s and A'_s for given permissible stresses. This case occurs in practice when a definite concrete section has to be used for aesthetic, space or other requirements. It is clear that it can be extended to cover the case in which the depth is not limited if the design is to be carried out by trial. Generally an approximate assumption for the depth of the section in the case of eccentric forces can be made by taking the depth required for simple bending as a basis, adding to it, say, 0 to 20 percent in the case of eccentric com-

*Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete submitting "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete," American Concrete Institute and others (1940).

pression, or subtracting from it 0 to 20 percent in the case of eccentric tension, provided that in both cases the force lies beyond the reinforcement.

Determination of A_s and A'_s

Consider the cases shown in Fig. 1 a, b and c, in which a rectangular section is subjected to simple bending, eccentric compression and eccentric tension respectively. The following equations apply for the three cases:

Taking moments about the tensile and compressive reinforcement respectively we get:

$$M_s = Pe_s = \frac{bf_c z}{2}\left(d - \frac{z}{3}\right) + nA'_s f_{c1}\,(d - d_1) \quad \text{(a)}$$

$$M'_s = Pe'_s = A_s f_s\,(d - d_1) - \frac{bf_c z}{2}\left(\frac{z}{3} - d_1\right) \quad \text{(b)}$$

But $z = \dfrac{nfc}{nfc + f_s}\,d = \dfrac{1}{1 + r}\,d = kd$

where $r = \dfrac{f_s}{nf_c}$

and $f_{c1} = f_c \dfrac{z - d_1}{z} = f_c\left(1 - \dfrac{i}{k}\right)$

where $i = \dfrac{d_1}{d}$

Substituting in (a), (b) and reducing we get:

$$\frac{M_s}{bd^2 f_c} = \frac{k}{2}\left(1 - \frac{k}{3}\right) + np'\left(1 - \frac{i}{k}\right)(1 - i)$$

$$\frac{M'_s}{bd^2 f_c} = np\,r\,(1 - i) - \frac{k}{2}\left(\frac{k}{3} - i\right)$$

From which

$$np' = \left(\frac{M_s}{bd^2 f_c}\right)\left(\frac{k}{(k - i)\,(1 - i)}\right) - \left(\frac{k^2(3 - k)}{6(k - i)\,(1 - i)}\right) \quad \text{(1)}$$

$$np = \left(\frac{M'_s}{bd^2 f_c}\right)\left(\frac{1}{r\,(1 - i)}\right) + \left(\frac{k\,(k - 3i)}{6r\,(1 - i)}\right) \quad \text{(2)}$$

Eq. (1) and (2) can be put in the following form:

$$np' = \left(\frac{M_s}{bd^2 f_c}\right) C_1 - C_2 \quad \text{(1a)}$$

$$np = \left(\frac{M'_s}{bd^2 f_c}\right) C_3 + C_4 \quad \text{(2a)}$$

Taking $d_1 = 0.08d$, *i.e.*, $i = 0.08$, Table 1 gives the values of C_1, C_2, C_3

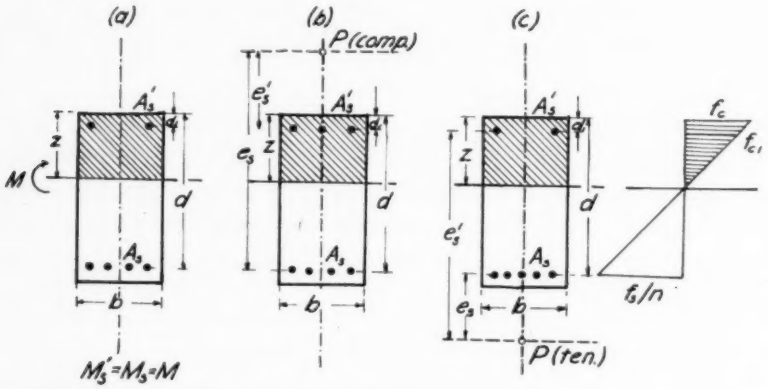

Fig. 1—Rectangular sections under simple bending, eccentric compression and eccentric tension

and C_4 for different values of $\frac{f_s}{nf_c}$. The curves in Fig. 2 give the values of np' and np directly.

Notice that in the case of simple bending $M'_s = M_s = M$.

Example: Find the required reinforcement if a rectangular section 12 x 30 in. is subjected to: (a) a bending moment of 300,000 ft lb, (b) same bending moment plus an axial compressive force of 40,000 lb, and (c) same bending moment plus an axial tensile force of 40,000 lb. Take $f_c = 1350$ psi, $f_s = 20{,}000$ psi and $n = 10$, concrete cover = 2 in.

$$\frac{f_s}{nf_c} = \frac{20{,}000}{10 \times 1350} = 1.48$$

(a) $$\frac{M_s}{bd^2f_c} = \frac{M'_s}{bd^2f_c} = \frac{300{,}000 \times 12}{12 \times 28^2 \times 1350} = 0.283$$

From Fig. (2): $p = 2.2$ percent, $p' = 1.55$ percent.

(b) Eccentricity from center line $= \frac{300{,}000 \times 12}{40{,}000} = 90$ in.

$e_s = 90 + 13 = 103$ in.

$e'_s = 90 - 13 = 77$ in.

$$\frac{M_s}{bd^2f_c} = \frac{40{,}000 \times 103}{12 \times 28^2 \times 1350} = 0.323$$

$$\frac{M'_s}{bd^2f_c} = \frac{40{,}000 \times 77}{12 \times 28^2 \times 1350} = 0.243$$

From Fig. (2): $p = 1.85$ percent, $p' = 2.0$ percent

(c) Eccentricity from center line $= \frac{300{,}000 \times 12}{40{,}000} = 90$ in.

$(d_1 = 0{,}08d)$

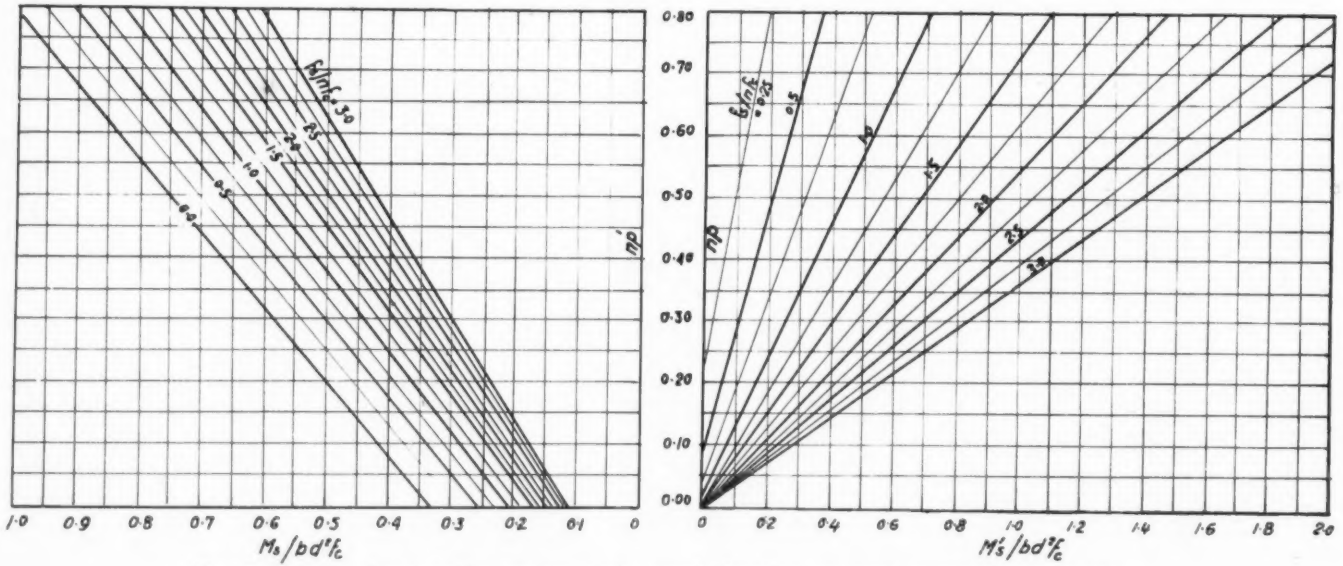

Fig. 2—Design of rectangular sections under simple bending, eccentric compression or tension

$e_s = 90 - 13 = 77$ in.

$e'_s = 90 + 13 = 103$ in.

$$\frac{M_s}{bd^2f_c} = 0.243 \quad , \quad \frac{M'_s}{bd^2f_c} = 0.323$$

From Fig. (2): $p = 2.40$ percent, $p' = 0.90$ percent

Table 1 could also be used instead of using Fig. 2.

Where d_1/d is very different from 0.08 it may be advisable to use Eq. (1) and (2), instead of Table 1 or Fig. 2, for a more accurate result.

Condition for minimum reinforcement

Since the dimensions of the concrete section are fixed, the most economical section is that which requires minimum total reinforcement.

In the cases of simple bending, eccentric tension, and eccentric compression with large eccentricity the minimum reinforcement is obtained if the maximum permissible stresses are fully utilized. However, for eccentric compressive forces with small eccentricities a lower value of f_s than the maximum permissible value may give less total reinforcement. The minimum reinforcement can be easily obtained from Fig. 2 by trying different values of $\frac{f_s}{nf_c}$ less than the permissible value and finding the corresponding $(A_s + A'_s)$. In the following analysis the exact value of f_s giving minimum reinforcement will be determined.

The condition for minimum reinforcement is that:

$$\frac{d}{df_s}(A_s + A'_s) = 0$$

$$\text{or } \frac{d}{df_s}(p + p') = 0.$$

Substituting for p and p' from Eq. (1) and (2) and differentiating we get:

$$n(1-i)\frac{d}{df_s}(p' + p) = \frac{M_s}{bd^2f_c}\left(\frac{-i}{(k-i)^2}\right)\frac{dk}{df_s} + \frac{M'_s}{bd^2f_c}\left(\frac{1}{(1-k)^2}\right)$$

$$-\left(\frac{3k^2-2k^3-6ik+3ik^2}{6(k-i)^2}\right)\frac{dk}{df_s} + \left(\frac{3k^2-2k^3-6ik+3ik^2}{6(1-k)^2}\right)\frac{dk}{df_s} = 0$$

Since $\frac{dk}{df_s} \neq 0$ we get:

$$\left(\frac{M_s}{bd^2f_c}\right) - \left(\frac{M'_s}{bd^2f_c}\right)\frac{(k-i)^2}{i(1-k)^2} +$$

$$\left(\frac{(3k^2-2k^3-6ik+3ik^2)\ (1-i)\ (1+i-2k)}{6i(1-k)^2}\right) = 0 \quad \ldots\ldots\ldots\ldots\ldots (3)$$

This equation gives the condition for minimum total reinforcement. Taking $i = 0.08$, the results are given in Fig. 3, from which f_s satisfying the condition for minimum reinforcement can be obtained. The values of p and p' can then be found from Fig. 2 or Table 1.

Example:

A section 12 x 30 in. is subjected to a couple of 200,000 ft-lb and an axial compressive force of 120,000 lb. If $f_c = 1350$ psi, $f_s = 20{,}000$ psi, $n = 10$ and concrete cover = 2 in., find minimum reinforcement: (a) by trial from Fig. 2, and (b) using Fig. 3.

$$\text{Eccentricity from center line} = \frac{200{,}000 \times 12}{120{,}000} = 20 \text{ in.}$$

$$e_s = 20 + 13 = 33 \text{ in.}$$

$$e'_s = 20 - 13 = 7 \text{ in.}$$

$$\frac{M_s}{bd^2 f_c} = \frac{120{,}000 \times 33}{12 \times 28^2 \times 1350} = 0.312$$

$$\frac{M'_s}{bd^2 f_c} = \frac{120{,}000 \times 7}{12 \times 28^2 \times 1350} = 0.066$$

(a) Maximum permissible $\dfrac{f_s}{nf_c} = \dfrac{20{,}000}{1350 \times 10} = 1.48$

Values of $\dfrac{f_s}{nf_c} = 1.48, 1.25, 1.0, 0.75, 0.5$ will be tried. The corresponding values of p and p' are obtained from Fig. 2.

$\frac{f_s}{nf_c}$	p, percent	p', percent	$(p+p')$, percent	
1.48	0.60	1.80	2.40	
1.25	0.70	1.55	2.25	minimum
1.00	0.95	1.30	2.25	minimum
0.75	1.40	0.97	2.37	
0.50	2.10	0.60	2.70	

The minimum reinforcement occurs for $\dfrac{f_s}{nf_c}$ between 1.0 and 1.25. The corresponding total reinforcement is $(p + p') = 2.25$ percent.

(b) From Fig. 3 we find that $\dfrac{f_s}{nf_c}$ corresponding to minimum reinforcement is 1.04. Then from Fig. 2:

$p = 0.90$ percent, $p' = 1.35$ percent, $p + p' = 2.25$ percent.

This corresponds to a tensile stress in steel

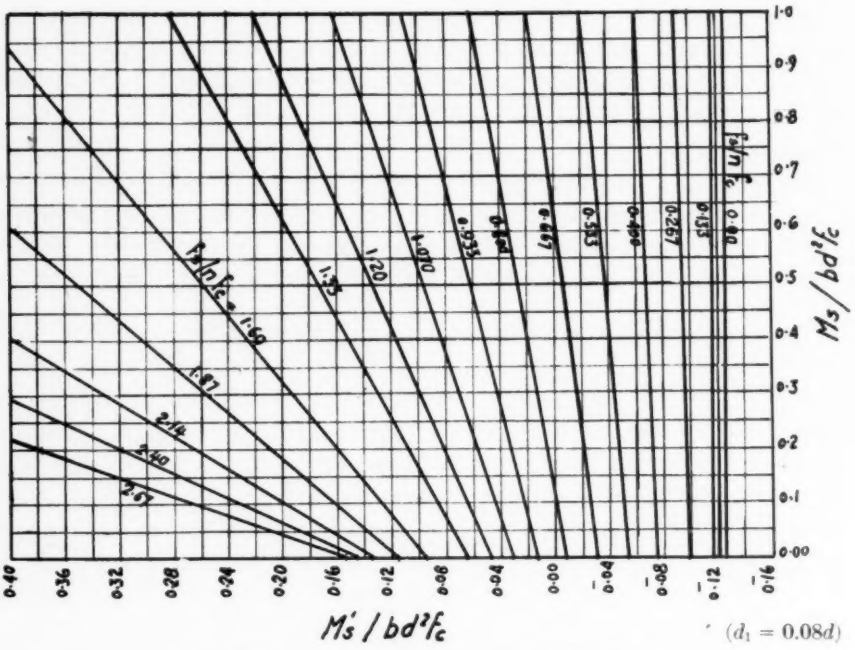

Fig. 3—Value of f_s giving minimum reinforcement

$f_s = 1.04 \times 10 \times 1350 = 14{,}050$ psi.

For smaller eccentricities the difference is more pronounced.

Notice that $(p + p')$ is not very sensitive to the variation in $\frac{f_s}{nf_c}$ and a choice within relatively wide range near the value giving minimum reinforcement may not influence the sum $(p+p')$ appreciably. Notice also that the importance of taking values of f_s lower than the maximum permissible value does not lie only in getting less total reinforcement but also in getting better distribution of tensile and compressive reinforcement conforming with the requirements of statics. (The effect of this change of steel distribution on the actual ultimate capacity of the section is beyond the scope of this paper.) Thus in this example, for $f_s = 20{,}000$ psi we get $p = 0.60$ percent and $p' = 1.80$ percent, while it can be seen from the table above that we can take $p = p' = 1.15$ percent for a lower value of f_s.

DESIGN WHEN ONLY THE WIDTH OF THE SECTION IS FIXED

Referring to Fig. 1, the eccentric force P in cases (b) and (c) can be resolved into a force P concentric with the tensile reinforcement and a couple $M_s = Pe_s$ about the tensile reinforcement. (For case (a) of

simple bending $P = 0$ and $M_s = M$.) Since P is concentric with the tensile reinforcement, then it permits a reduction of the tensile steel by $\frac{P}{f_s}$ in the case of eccentric compression and requires an increase of the tensile steel by $\frac{P}{f_s}$ for eccentric tension. Except for that, the dimensioning for the couple M_s can be dealt with in the same way for the three cases.

Let the required depth for resisting M_s alone be d and the required reinforcement be A_{s1}, on the tension side and A's on the compression side. Assume that we choose $A'_s/A_{s1} = \alpha$. From the conditions of equilibrium the following equations apply for the three cases:

$$A_{s1}f_s = \frac{bf_c z}{2} + nf_{c1}A'_s \quad \text{(c)}$$

$$M_s = \frac{bf_c z}{2}(d - z/3) + nf_{c1}A'_s(d-d_1) \quad \text{(d)}$$

But $z = \frac{1}{1+r}d = kd$, where $r = \frac{f_s}{nf_c}$

and $f_{c1} = f_c\left(1 - \frac{i}{k}\right)$, where $i = \frac{d_1}{d}$

Substituting in (c) and (d) we get

$$np_1 = \frac{k}{2\left\{r - \alpha\left(1 - \frac{i}{k}\right)\right\}} \quad \text{(4)}$$

then $A'_s = \alpha A_{s1}$

and $M_s = Rbd^2$ (5)

where $R = Cf_c$ and $C = \frac{k}{2}\left(1 - \frac{k}{3}\right) + \alpha np\left(1 - \frac{i}{k}\right)(1-i)$

Then the net area of steel A_s is given by

$$\left.\begin{aligned} A_s &= A_{s1} && \text{for simple bending} \\ A_s &= A_{s1} - \frac{P}{f_s} && \text{for eccentric compression} \\ A_s &= A_{s1} + \frac{P}{f_s} && \text{for eccentric tension} \end{aligned}\right\} \quad \text{(6)}$$

For any chosen ratio α, the values of d, A_s and A'_s can be obtained from Eq. (4), (5) and (6). Taking $d_1 / d = 0.08$, the curves shown in Fig. 4 give the depth and reinforcement for ratios $\alpha = 0$, 0.2, 0.4, 0.6,

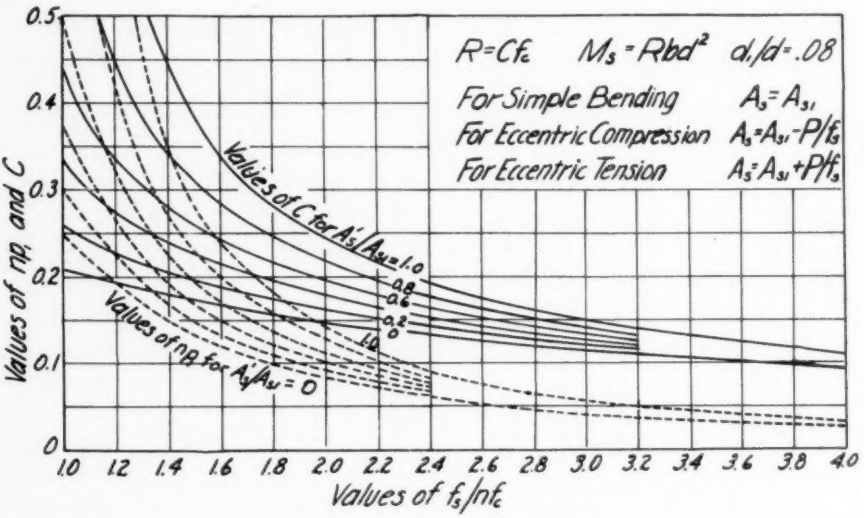

Fig. 4—Design of rectangular sections under simple bending, eccentric compression and eccentric tension

0.8, 1.0. Notice that $\alpha = A'_s / A_s$ only for simple bending. For eccentric forces $\alpha = A'_s / A_{s1}$*. Notice also that for computing M_s a preliminary assumption of d should be made. One repetition may be required in some cases.

For eccentric compressive forces at small eccentricities (say within an eccentricity from the center line equal to the depth of the section), it is advisable to try values of f_s lower than the maximum permissible value. Fig. 3 may be consulted for that purpose.

Example 1: Find the required depth and reinforcement of a rectangular section 12 in. wide to carry: (a) a bending moment of 300,000 ft-lb, (b) same bending moment plus an axial compressive force of 40,000 lb and (c) same bending moment plus an axial tensile force of 40,000 lb. Take $f_c = 1350$ psi, $f_s = 20{,}000$ psi and $n = 10$. Use $A'_s/A_{s1} = 0.4$.

$$\frac{f_s}{nf_c} = \frac{20{,}000}{10 \times 1350} = 1.48$$

(a) From Fig. 4 $np = 0.177$, $R = 0.23 \times 1350 = 310$ psi
From Eq. (5) $300{,}000 \times 12 = 310 \times 12 \times d^2$, d = 31 in.

Thus $$A_s = \frac{0.177}{10} \times 12 \times 31 = 6.6 \text{ in.}^2$$

$$A'_s = 0.4 \times 6.6 = 2.64 \text{ in.}^2$$

(b) From Fig. 4 $np = 0.177$, $R = 310$ psi

*For curves giving the depth and reinforcement for eccentric forces in terms of A'_s/A_s, see Bulletin 2—Reinforced Concrete Section, Faculty of Engineering, Fouad I University Giza, Egypt.

Assume $t = 36$ in. and cover $d_1 = 2.0$ in.

$$e = \frac{300{,}000 \times 12}{40{,}000} = 90 \text{ in.}$$

$$M_s = 40{,}000\ (90 + 16) = 4{,}240{,}000 \text{ in.-lb.}$$

From Eq. (5) $4{,}240{,}000 = 310 \times 12 \times d^2$, d = 33.5 in.

Then $$A_{s1} = \frac{0.177}{10} \times 12 \times 33.5 = 7.1 \text{ in.}^2$$

$$A_s = 7.1 - \frac{40{,}000}{20{,}000} = 5.1 \text{ in.}^2$$

$$A'_s = 0.4 \times 7.1 = 2.84 \text{ in.}^2$$

(c) From Fig. 4 $np = 0.177$, $R = 310$ psi

Assume $t = 31$ in. and cover $d_1 = 2.0$ in.

$$e = \frac{300{,}000}{40{,}000} \times 12 = 90 \text{ in.}$$

$$M_s = 40{,}000\ (90 - 13.5) = 3{,}060{,}000 \text{ in.-lb.}$$

From Eq. (5) $3{,}060{,}000 = 310 \times 12 \times d^2$, $d = 28.6$ in.

Then $$A_{s1} = \frac{0.177}{10} \times 12 \times 28.6 = 6.1 \text{ in.}^2$$

$$A_s = 6.1 + \frac{40{,}000}{20{,}000} = 8.1 \text{ in.}^2$$

$$A'_s = 0.4 \times 6.1 = 2.44 \text{ in.}^2$$

Example 2: In the preceding example, find the required depth and reinforcement for case (a) using ratios of A'_s/A_s equal to 0, 0.2, 0.4, 0.6, 0.8, 1.0.

The required values can be directly obtained from Fig. 4. The results are as follows:

A'_s/A_s	0	0.2	0.4	0.6	0.8	1.0
d, in.	35.6	33.6	31.4	29.0	26.5	23.7
A_s, in.^{2}	5.8	6.18	6.56	7.05	7.66	8.45
A'_s, in.^{2}	0	1.24	2.62	4.23	6.13	8.45

Cases (b) and (c) could also be dealt with in the same way.

On the above basis of computation, it will generally be found that the lower the ratio of compressive reinforcement, the lower is the cost per linear foot at the design section. But the economy of the structure can not be considered in terms of the cost at the critical position of each element alone. Nor even can it be considered in terms of the least expense for each complete element separately. It should be studied

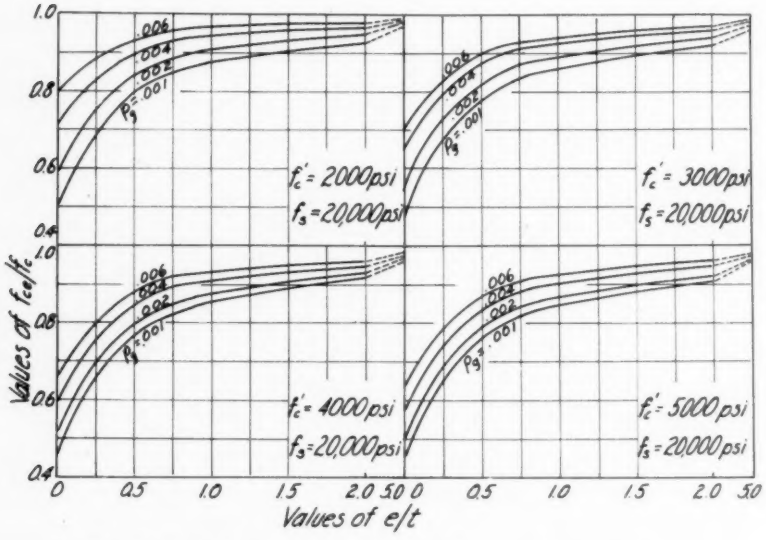

Fig. 5—Reduction of f_c to f_{ce} in case of eccentric compression

for the structure as a whole, taking into account such factors as the effect on the weight and height of the structure. The use of the given curves permits a flexibility in design by making it possible to find out many alternatives in a relatively short time.

ADAPTATION TO THE 1940 JOINT COMMITTEE SPECIFICATIONS*

The following few points need be considered in order to adapt the general methods described before to the specifications of the 1940 Joint Committee.

Value of $\frac{f_s}{nf_c}$

According to the specifications $f_c = 0.45f'_c$ and the value of n for concrete made of normal weight aggregate can be considered as given by $n = \frac{30{,}000}{f'_c}$. Thus nf_c is a fixed value equal to 13,500 psi. Taking $f_s = 20{,}000$ psi we get:

$$\frac{f_s}{nf_c} = 1.48 \quad (7)$$

and $k = 0.404$ (8)

Substituting these values in the preceding design formulas, and taking $d_1/d = 0.08$ we find that:

*See footnote p. 382.

(a) When the depth is fixed, the required areas of compressive and tensile reinforcement for rectangular sections subjected to simple bending, eccentric compression or eccentric tension are given by:

$$np' = 1.35 \frac{M_s}{bd^2 f_c} - 0.238 \quad \ldots\ldots (9)$$

$$np = 0.735 \frac{M'_s}{bd^2 f_c} + 0.008 \quad \ldots\ldots (10)$$

It is advisable, however, in the case of eccentric compression at small eccentricities, to try values of f_s below 20,000 psi using Fig. 2.

(b) When the depth is not fixed, then for any fixed ratio A'_s/A_{s1}, the depth and reinforcement can be obtained from Table 2 for simple bending as well as for eccentric forces.

Values of f_c for eccentric compression

The specifications require that a reduced value for the concrete fiber stress in compression should be used in the case of eccentric compression. Its value is given by:

$$f_{ce}^* = f_a \frac{1 + \frac{ec}{R^2}}{1 + C\frac{ec}{R^2}} \quad \ldots\ldots (11)$$

where for tied columns $f_a = 0.8 \dfrac{0.225 f'_c + f_s p_g}{1 + (n-1)\, p_g}$ and $C = \dfrac{f_a}{0.45 f_c}$

Taking $\dfrac{ec}{R^2}$ as approximately equal to $\dfrac{6e}{t}$, the curves shown in Fig. 5 give the values of f_{ce} for different values of f'_c and p_g. The reduction at large eccentricities, say $e/t > 2$, may be neglected without a serious error. For smaller eccentricities the reduced value of f_c should be used.

Reduction in compression reinforcement

The specifications permit a reduction in the value of the compression reinforcement A'_s obtained from the theory as follows:

$$\text{Compression reinforcement} = \frac{nf_c\,(k-i)}{16{,}000} A'_s \not< \tfrac{1}{2} A'_s \quad \ldots\ldots (12)$$

where, according to the specifications $nf_c = 13{,}500$ psi and $k = 0.404$ i is the ratio of cover to effective depth.

SUMMARY

When the width and depth of the rectangular section are fixed, Fig.

*The term used in the specifications is f_c. The rest of the notation is the same.

2 gives the tensile and compression reinforcement for the cases of simple bending, eccentric compression and eccentric tension for any values of f_s, f_c and n. Table 1 may also be used for the same purpose. In the case of eccentric forces with small eccentricities, values of f_s lower than the maximum permissible stress may lead to less total $A_s + A'_s$ and better distribution of steel. Fig. 3 gives the value of f_s corresponding to minimum reinforcement.

TABLE 1—VALUES OF C_1, C_2, C_3 AND C_4

f_s/nf_c	0	0.25	0.50	0.75	1.00	1.25	1.50	1.75	2.00	2.25	2.50	2.75	3.00
C_1	1.18	1.20	1.23	1.26	1.29	1.32	1.36	1.39	1.43	1.47	1.51	1.55	1.61
C_2	0.394	0.354	0.318	0.293	0.268	0.250	0.234	0.222	0.212	0.203	0.195	0.188	0.183
C_3	—	4.35	2.18	1.45	1.09	0.873	0.726	0.622	0.545	0.484	0.436	0.396	0.364
C_4	—	0.325	0.103	0.046	0.023	0.013	0.008	0.005	0.003	0.002	0.001	0.0005	0.0003

$$np' = \left(\frac{M_s}{bd^2 f_c}\right) C_1 - C_2 \quad , \quad np = \left(\frac{M'_s}{bd^2 f_c}\right) C_3 + C_4$$

When only the width of the section is fixed, the dimensioning for all the cases mentioned before can be carried out from Fig. 4 for any chosen ratio A'_s/A_{s1}.

If we take $f_c = 0.45\ f'_c$, $n = \dfrac{30{,}000}{f'_c}$ and $f_s = 20{,}000$ psi, then the design under simple bending, eccentric compression and eccentric tension can be carried out for any concrete strength by the use of Eq. (9) and (10) when the depth of the section is fixed and by the use of Table 2 when it is not fixed. The reduction in f_c for sections subjected to eccentric compression as required by the 1940 Joint Committee specifications can be obtained from Fig. 5. Besides, the specifications permit a reduction of the compression steel in all cases as given by Eq. (12).

TABLE 2—VALUES OF p AND R

A'_s/A_{s1}	0	0.20	0.40	0.60	0.80	1.0
np	0.136	0.153	0.174	0.202	0.241	0.297
C	0.174	0.196	0.225	0.263	0.316	0.393

$R = Cf_c$, $M_s = Rbd^2$,

$A_s = A_{s1}$ For simple bending

$A_s = A_{s1} - \dfrac{P}{f_s}$ For eccentric compression

$A_s = A_{s1} + \dfrac{P}{f_s}$ For eccentric tension

When the foreman on a Lapland construction job removed the wooden chairs supporting compression steel, the bars sank in the green concrete as much as 2½ in. out of design position. To avoid costly stripping of the finished bridge floor, additional reinforcement was inserted at the proper level by grooving the concrete and grouting rods into the slots.

Strengthening Bridge Slabs with Grouted Reinforcement*

By S. O. ASPLUND†

SYNOPSIS

On the project described the negative reinforcing bars settled as much as 2½ in. out of correct position. Various means for incorporating negative reinforcement at correct height in the finished structure are discussed. The method selected of grouting additional bars in grooves cut with the aid of diamond saw is described. Tests on beam specimens made with some bars encased in the concrete and some bars grouted into diamond-sawed grooves, give results identical with those to be expected for all bars normally encased; ultimate strengths conform closely to the plastic bending capacity of reinforced concrete according to design methods originated by C. S. Whitney.

REINFORCING BARS SINK IN FRESH CONCRETE

In the construction of a concrete bridge deck in Lapland (Fig. 1) the upper reinforcement bars were mounted on wooden chairs. Immediately after the concrete was poured the foreman pulled these chairs out of the concrete and finished the surface to the exact height demanded by the drawings. He did not notice that the reinforcing bars proceeded to sink through the lighter, still green concrete. Holes and grooves were later chiseled in the surface of the concrete to determine the position of the bars. In 160 ft of completed bridge deck the upper reinforcement was found to have sunk distances up to 2½ in.

REPAIR METHODS CONSIDERED

The bridge floor was not acceptable in that condition. A costly stripping of concrete and reconstruction seemed unavoidable. New cement and reinforcement would have to be used despite shortage of

*Received by the Institute March 22, 1948. Title 45-21 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 5, Jan. 1949, *Proceedings* V. 45. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than May 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

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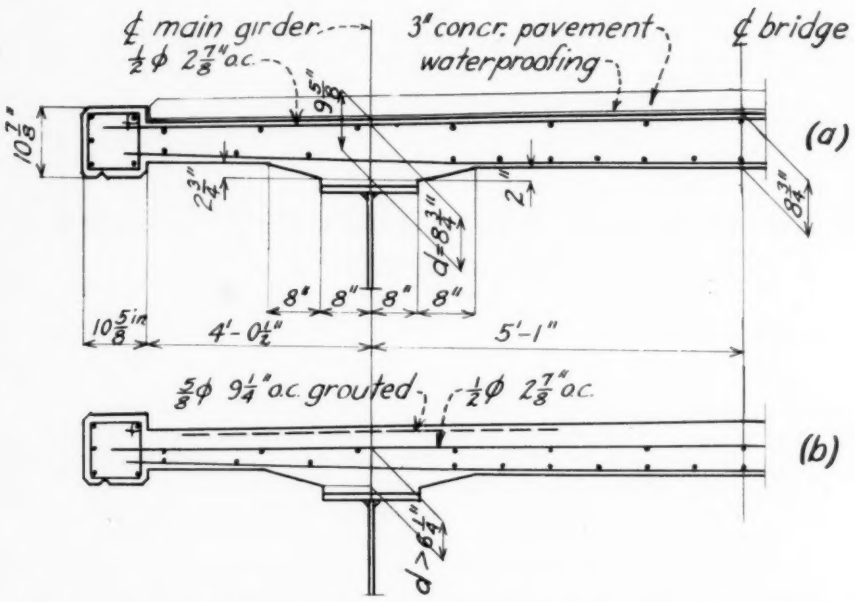

Fig. 1—Half section of bridge floor as planned is shown in (a). In (b) the defective slab with sunken reinforcement is indicated. Dash line shows grouted rods for repair.

these materials. Such unpleasant prospects prompted seeking other ways out of this dilemma.

Would it be possible in some manner to put down additional bars in the bridge slab? Grouting such bars in chiseled grooves in the concrete could be neither neat nor satisfactory. When the high quality concrete of the deck was chiseled the concrete probably would break irregularly. The chiseling procedure might also cause fine cracks in the concrete close to the grooves whereby the necessary interaction of concrete and reinforcement would be endangered. It would be possible to determine by experiments and tests the efficiency of bars put down in this manner. However, low strengths with a large dispersion might be expected, necessitating low working loads. These prospects were not encouraging, and the idea of grouting complementary bars in chiseled grooves was therefore abandoned.

Ground grooves would make a neater and more exact job without damage to the adjacent concrete. Limestone was known to be quarried by carborundum saws, and conferences with a manufacturer of carborundum tools confirmed the possibility of grinding in concrete. With a carborundum saw powered by at least a 20 hp motor, grooves probably could be ground in the bridge deck concrete. The forward feed of the machine would have to be automatic, and sidesway could not be

very large. However, the carborundum discs would wear out quickly and the estimated cost of depreciation necessitated giving up this project also.

To grind with carborundum two *thin* slits at a distance of the diameter of the reinforcing bars would cost less but was out of the question. The discs would break, even if the sidesway of the grinding machine were kept very small, and such a sidesway could hardly be eliminated. The thin steel plate of a circular saw would not break so easily as a carborundum disc. Would it be possible to grind such slits with a circular saw set with diamonds? Test cylinders are often taken from concrete structures in the form of diamond-drilled cores. If two parallel slits were ground $\frac{3}{4}$ in. apart, the concrete between could be cut away with a chisel and the bars laid in place. In this way the volume of concrete ground away could be reduced to a fraction of that necessary when using carborundum tools.

DIAMOND SAW SELECTED

No experience with diamond-sawing of concrete was available but rough estimates of wear pointed to economic feasibility, so a machine saw was designed and built. A shaft carrying the diamond circular saw was driven by a 7.5 hp electric motor. This arrangement was hinged to a chassis in such a manner that the saw could be lowered or raised by a screw with a crank. Two grooved wheels of the chassis straddled exactly the up-turned back of a steel angle. The two other wheels ran freely on the back of a steel channel. The carriage was fed forward by hand. To obtain an even feed the operator pushed the machine forward so as to keep a constant reading on an ammeter connected to the motor.

The grinding saw was a $\frac{5}{64}$-in. steel plate 12 in. in diameter. Along its rim was a coating, $\frac{1}{8}$ in. thick and $\frac{1}{4}$ in. high, of metal-bound diamonds. During operation the saw was cooled with a water jet to prevent destruction by heat.

The machine was tested on an ordinary concrete floor and worked satisfactorily. The problem of making the grooves could be considered solved.

SLAB TEST PROGRAM

Now the question remained—to what extent would the grouted bars act with the concrete floor slab. The concrete deck in Fig. 1 was designed for a moment M of 12,700 in.-lb per in. over the main steel girders and for a shear T of 566 lb per in. in the cantilevered deck near the main girder. If a test slab, Fig. 2, is loaded with 16,300 lb at the center there

will be a shear of 8150 lb or 566 lb per in. and a moment of 8150 × 23.6 = 192,000 in.-lb or 13,300 in.-lb per in. Hence moment and shear are comparable to those in the loaded bridge. Other conditions in the test slab are more unfavorable: the maximum moment occurs at the same time as the shear; the narrow slab does not (as in the bridge) act jointly with other parts of the deck; and finally, the anchorage length of the reinforcement bars is considerably smaller than in the bridge.

The test slab, Fig. 2, has the same depth and reinforcement as the defective bridge deck above the supports. If an additional 0.89 sq in. of reinforcement is placed in slots at a distance of 8.7 in. from the outermost compression fiber and a specified working stress of 27,000 psi in the deformed bars (specified yield point 57,000 psi) and 850 psi in the concrete is applied, an allowable moment of 12,700 in.-lb per in. may be computed according to the plastic theory.

For the bridge slab a high grade concrete with 505 lb of standard cement per cu yd and a 28-day ultimate cube strength of 4620 psi was demanded (cylinder strength about 4000 psi). Six preliminary test cubes poured June 6, 1946, averaged 6260 psi, and four sets of three cubes each, poured on Oct. 12, and 22, Nov. 23 and Dec. 3, 1946, yielded averages of 5560, 6220, 5390, and 6150 psi, respectively, or a mean value of 5830 psi. The defective bridge floor with the dislocated reinforcement was poured under favorable weather and working conditions. No honeycombing or other surface defects were observed after removal of forms. A cube strength of at least 5830 psi may be assumed, or a cylinder strength of 0.85 × 5830 = 4950 psi or a compressive strength of 0.85 × 4950 = 4200 psi in the completed structure.

If the floor section as planned had been designed according to the plastic theory, Fig. 3, an ultimate bending moment of 3480 lb per in. × 8.33 in. = 28,900 in.-lb per in. could have been expected. In the defective bridge floor where the effective depth 8.74 in. was reduced to 6.3 in., only 3740 lb per in. × 5.89 in. = 22,000 in.-lb per in. may be counted. If 0.019 sq in. per in. is placed in slots in the latter slab on

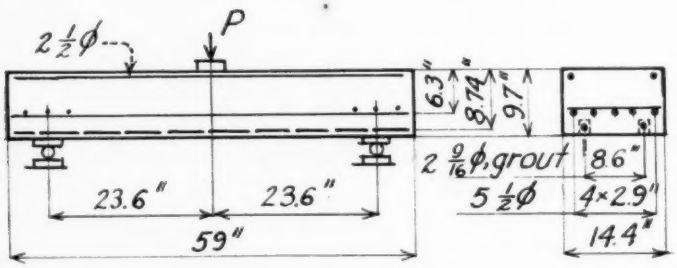

Fig. 2—Test specimen

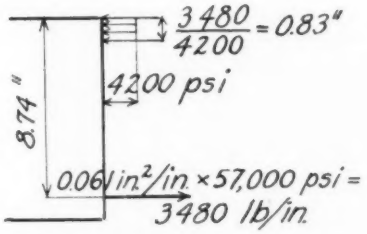

Fig. 3—Failure stresses in deck as planned

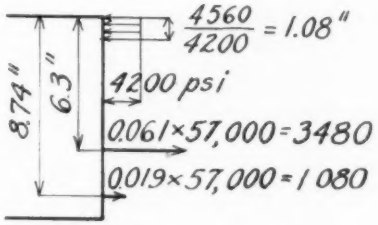

Fig. 4—Failure stresses in proposed repair

the effective depth 8.74 in. (Fig. 4) the ultimate moment may be calculated as $1080 \times 8.20 + 3480 \times 5.76 = 28{,}900$ in-.lb per in., or the same moment as was originally intended.

The authorities did not accept this computation. They demanded the floor slab to be designed according to the classic theory using a linear strain diagram and Hooke's law. Because of this a reinforcement of 0.0334 sq in. per in. had to be added to the slab instead of 0.019 sq in. per in. Then, on the other hand, an ultimate bending moment (Fig. 5) of $1900 \times 8.10 + 3480 \times 5.66 = 35{,}090$ in.-lb per in. or 505,000 in.-lb on a width of 14.4 in. could be computed. A similar computation based on a concrete ultimate strength of 5000 instead of 4200 psi and yielding 37,700 in.-lb per in. was presented to the owner before the test specimens were poured.

On June 11, 1947, three test slabs as shown in Fig. 2 were poured at the bridge site with deformed reinforcing bars from the same shipment used in the defective bridge floor (specified yield point $\geqq$ 57,000 psi). The quantities and positions of the bars were the same as in the defective bridge floor. By official tests the $\frac{15}{32}$-in. bars were found to have an average cross sectional area, longitudinal fins included, of 0.174 sq in., a lower yield point of 66,500 psi (considerably more than the specified lower limit) and an elongation on 10 diameters of 20.4 percent. The cement, the aggregates, the proportioning 1:2.92:3.54 by weight, and the slump of the concrete were the same as used when pouring the bridge floor and the above-mentioned sets of test cubes. The test slabs were stored moist from June 11 to June 29. On July 5

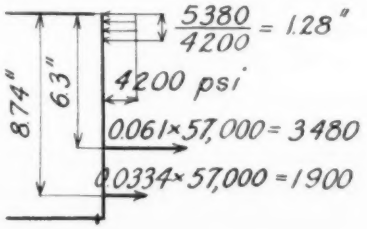

Fig. 5—Failure stresses in approved repair project

Fig. 6—Diamond sawing of test slabs

four slits 1.4 in. deep were diamond-sawed in each slab (Fig. 6) and the ¾ in. wide grooves chiseled out. The grooves were closed at their ends and filled with water from July 5 to July 9. It was thought that the grooves would be widened by the swelling of the concrete. When the concrete dried afterward it would shrink and jam the mortar cast into the grooves.

The question arose whether the smooth diamond-sawed sides of the grooves needed any roughening. A cement laboratory contended that no roughening was needed. Despite this the owner asked for roughening as a precautionary measure. After removing the water vertical furrows ⅛ in. deep and ⅛ in. wide were cut ¾ in. apart in the vertical sides of the grooves (with a rawl plug drill No. 10). The appearance of the furrows varied somewhat depending upon whether they were chiseled in a mortar or stone surface. Only once a stone loosened, leaving a hole about ⅜ in. in diameter. An investigation as to whether such roughening is really needed would be of theoretical and practical interest.

A wet mortar made of one part of cement, one part of fine sand, and water was worked into the sides of the grooves with a small steel brush. Surplus mortar was removed. The reinforcing bars were then grouted into the grooves with a mortar of the same cement content as that used in the concrete of the test slabs, that is, of the proportions 1:2.9. The largest sand size was ⅜ in., the same as in the bridge floor and in the test slabs. The mortar was so dry that its surface moistened only after being worked. One third of the depth of the grooves was filled with mortar, and then the deformed bars were laid down with their transverse fins towards the sides of the grooves. The mortar was worked to a

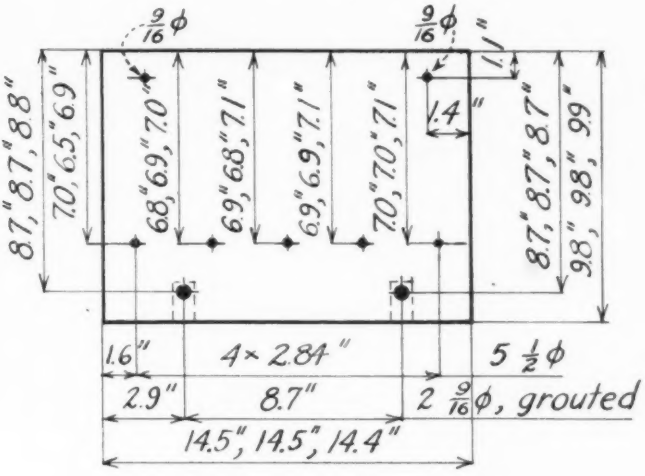

Fig. 7—Actual dimensions of test slabs

jelly consistency by knocking the bars. Then the groove was filled with mortar that was worked and leveled off. The grooves were covered with wet felt strips. Later sand was put on and sprinkled with water. The test slabs were stored moist for ten days and transported for official testing. Two weeks after grouting, they were accurately measured, loaded to failure, and then demolished for checking the position of all reinforcing bars. The official report of the measured positions is given by Fig. 7.

Based upon these measurements and the yield values 66,500 and 4200 psi, respectively, for the reinforcement and the concrete, the ultimate load of the test slabs can be figured by the plastic theory as 51,600, 51,600, and 52,900 lb, respectively.

Each test slab was supported on a rocker and a roller shoe as shown in Fig. 2 and 8 and loaded at its center with a line load uniformly distributed across the entire width of the slab. During the test the deflection under the load was measured by two indicator dials placed under the slab. At the same time cracks appearing in the slab were observed. The deflections during the tests are given in Table 1.

Fig. 8 shows the test slabs after failure with the cracks painted in black. On test slab No. 72,795 the cracks are numbered in the same order as they appeared. The primary cause of failure probably was shear in all three cases. When the slabs were demolished after the tests it was observed that the bond between the grouted reinforcing bars and the mortar in the grooves, and that between the mortar and the concrete of the slab, had been intact and that no slips had taken place.

TABLE 1—TEST SLAB DEFLECTIONS*

Load P, lb	Deflection at P, thousandths of in.		
	Slab No. 72,793	Slab No. 72,794	Slab No. 72,795
2 205	4	5	4
3 087	5	6	5
4 190	9	11	11
5 292	14	16	14
6 395	17	19	17
8 600	22	23	21
10,805	29	29	27
13,010	34†	34	32
15,215	39	39†	35†
17,420	46	46	41
19,625	53	52	46
22,932	61	63	55
26,240	72	74	65
29,547	82	84	73
32,855	93	95	84
37,265	113	109	96
41,675	132	126	111
49,172	—‡	—	—
52,700		—‡	—
55,125			—‡

*All deflections are recorded as the mean of two observations.
†First visible tension crack in concrete ‡Ultimate load

The ultimate loads 49,172, 52,700 and 55,125 lb coincide with those computed according to the plastic theory (51,600, 51,600, and 52,900 lb). Thus the tests satisfactorily verified that the computations submitted for the repair of the bridge floor were applicable.

BRIDGE REPAIRS

Now 4100 ft of slits were diamond sawed in the bridge floor (Fig. 9). The grooves were chiseled out, roughened and dampened in the same way as in the test slabs. Finally 2000 ft of deformed bars were grouted (Fig. 10) as in the test slabs, except for brushing with mortar since that now was considered superfluous by the owner. On March 11, 1948, after loading with trucks having 3½ ton wheel loads, the bridge was finally accepted by the owner.

CONCLUSIONS

The tests and measures performed provide a basis for the following summarizing observations and conclusions:

1. With a diamond circular saw it is practical and economical to saw narrow slits in a concrete surface in such a way that the space between two slits may be chiseled out to form grooves where reinforcing rods may be grouted in place. Another possible use for such grooves is for installing pipes or other conduits in hardened concrete floors.

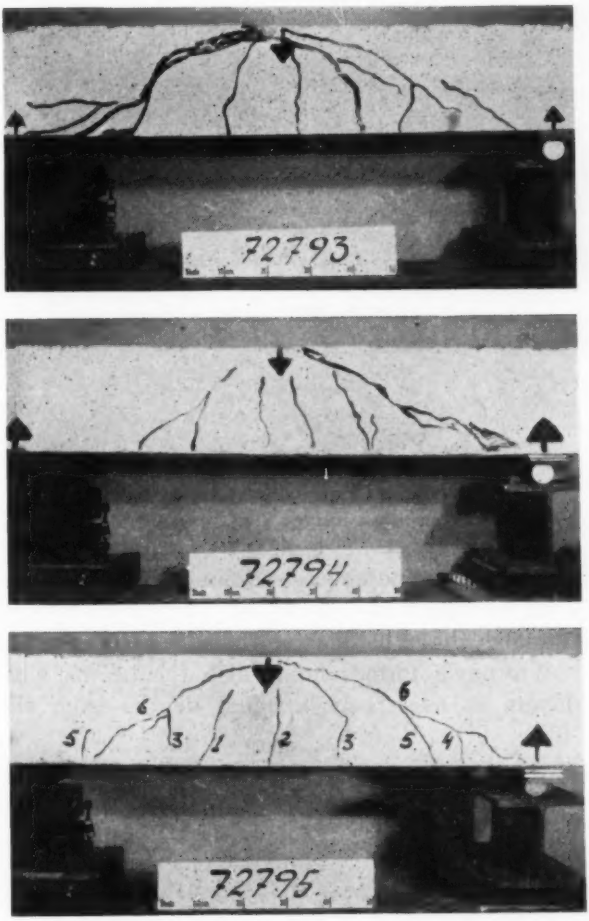

Fig. 8—Test slabs after failure

2. The diamond saw is not damaged or excessively worn if it happens to touch or saw through a reinforcing bar.

3. Before the grouting, the sides of the grooves should be saturated with water, for example by filling the grooves with water.

4. The reinforcement should be grouted with mortar of the same proportions as in the concrete structure. It should be just wet enough to become jelly-like after working.

5. Provided items 3 and 4 are followed, grouted reinforcing bars can be considered to be equivalent to cast-in-place reinforcement. The test slabs of this paper were designed to be about equally resistant to bending and shear failure. The primary cause of failure probably was shearing failure very close to bending failure. No tendency to failure by sliding of the reinforcement could be observed, notwith-

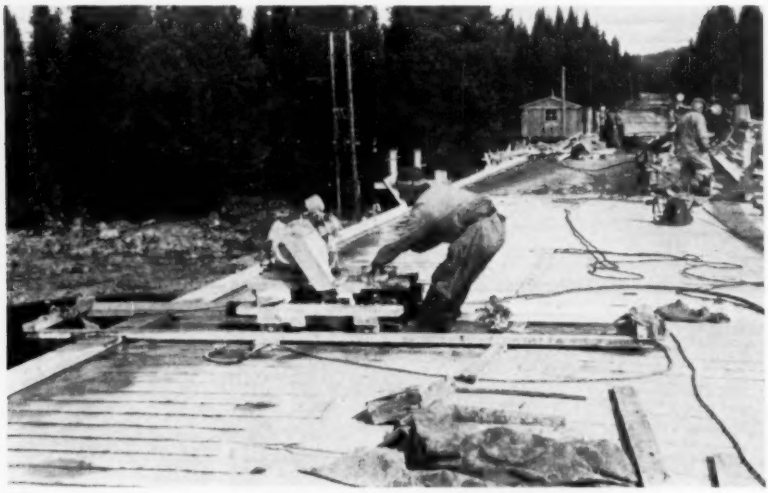

Fig. 9—Diamond sawing of bridge floor

standing the fact that the shear in the concrete around the reinforcing bars (the bond) was figured as high as about 400 psi.

6. The tests have verified that the plastic theory of concrete in flexure, particularly in Whitney's form*, holds also for beams with reinforcement in two layers at essentially unequal depths from the outermost compression fibre.

*Whitney, C. S., "Plastic Theory of Reinforced Concrete Design," *Proceedings* A.S.C.E., Dec. 1940, p. 1749.

Fig. 10—Grouting of deformed bars

Title No. 45-22

More economical sections in rigid concrete frames with secondary stresses of less consequence are possible, according to Mr. Brumer, when the plastic theory is used in design.

Comparative Designs of a Segmental Skewed Frame Concrete Bridge by the Straight Line and Plastic Theory Methods*

By MILTON BRUMER†

SYNOPSIS

The designs of a segmental skewed frame concrete bridge are discussed and compared as developed by both the straight line and plastic theory methods. The plastic theory will lead to rigid concrete frames of more economical proportions capable of sustaining equal or greater loads than frames designed by the straight line method. Effects of volume changes and yielding supports are also of smaller consequence. Deflections may be expected to be greater. The author concludes that further substantiation of the plastic theory is desirable.

Standard design methods have resulted in the construction of right and skewed frame structures which experience has proved adequate from the point of view of safety. These methods, however, due to certain peculiarities of the frame, result in relatively heavy and expensive designs in relation to the vertical live loads carried. Utilization of the plastic theory in reinforced concrete design indicates that these heavy proportions can be lightened and yet provide equivalent capacity for all loads.

Present theoretical stress analysis of skewed frames, in the writer's opinion, leaves much to be desired as an accurate presentation of the behavior of the structure; however, in the absence of a better approach, these methods of stress analysis are accepted as the basis for the section design used herein for the comparison of the straight line and plastic theory methods.

*Received by the Institute Sept. 15. 1948. Title No. 45-22 is a part of copyrighted JOURNAL of the AMERICAN CONCRETE INSTITUTE, V. 20, No. 5, Jan. 1949, *Proceedings* V. 45. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than May 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

This paper was prepared by the author and is presented as part of the work of Committee 314, Rigid Frame Bridges, D. H. Pletta, chairman. Model tests of the frames considered in this paper are now underway.

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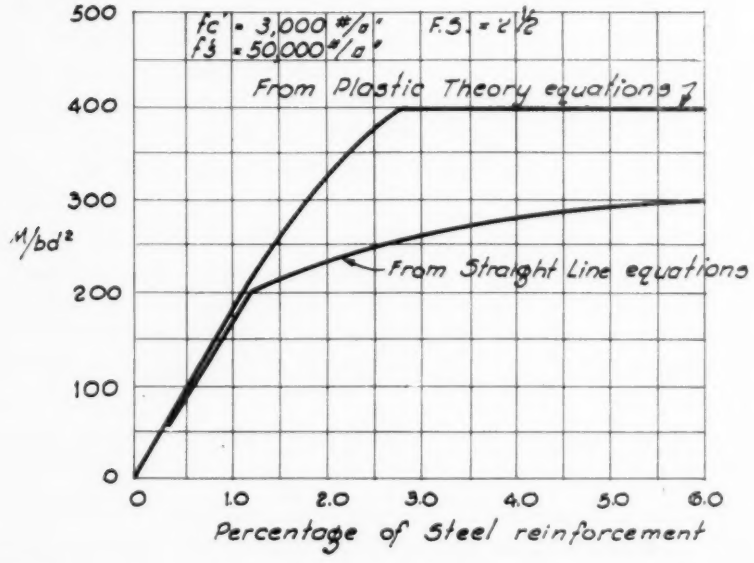
The plastic theory recognizes the plasticity of the concrete materials at or near failure and thus realistically anticipates the ultimate strength of the member. It also provides a more adequate estimate of the behavior of the concrete members under combined direct stress and flexure. Comparisons of a wide range of test values, as noted by Whitney,* for beams and for columns with loads at varying degrees of eccentricity are in close agreement with the computed strength obtained by use of the plastic theory.

Fig. 1 shows the possibility of greater variation in the values of width and depth of a member under the plastic theory than with the straight line theory at various percentages of reinforcement. This phenomenon is of particular importance in the crown region of the frame where the reinforcement may readily exceed 1 percent of the cross section. This means that the dead load, using the plastic theory, can be reduced throughout, and that other stresses such as those from temperature and shrinkage and yielding of supports are likewise reduced because of the consequent reduction in stiffness of members.

As used in this paper, the term "factor of safety" is the ratio of the ultimate thrust capacity of the section, as determined by the plastic theory, to the assumed working thrust acting at the same eccentricity;

Fig. 1—Comparison of resisting moments of rectangular beams calculated from straight line equations and plastic equations (no thrust).
Whitney, Charles S., Plastic Theory of Reinforced Concrete—*Proceedings* A.S.C.E., Dec. 1940.



*Whitney, Charles S., "Plastic Theory of Reinforced Concrete Design", *Transactions* A.S.C.E., V. 107, (1942). All references to the plastic theory are based on the relationships derived in this paper.

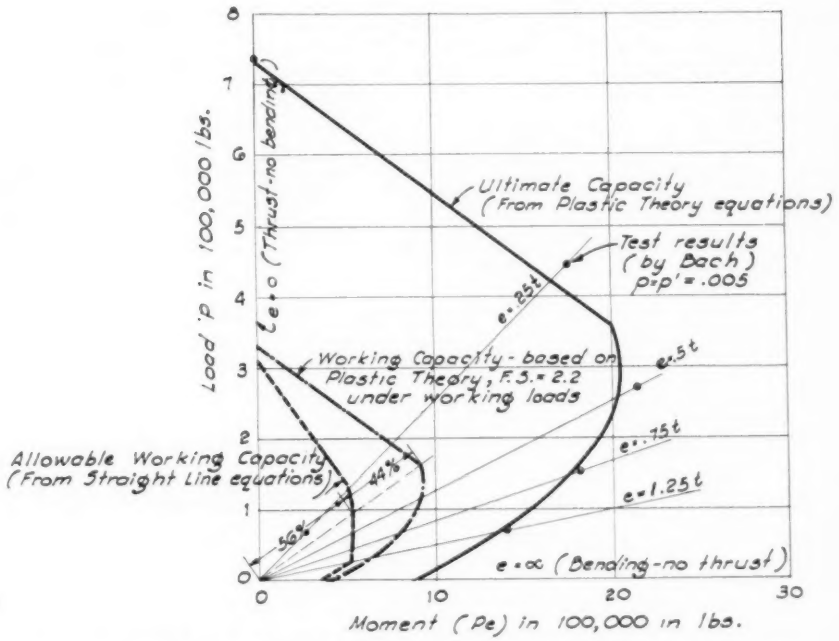


Fig. 2—Example of rectangular beams under combined bending and direct stress

Report of Committee 312, Whitney, Charles S., Chairman, "Plain and Reinforced Concrete Arches", ACI JOURNAL, Sept. 1940, Proc. V. 37, p 1.

or, in the case of pure flexure, the ratio of the ultimate bending moment to the bending moment caused by the assumed loading conditions. The "theoretical safety factor" is to be distinguished from the term "factor of safety", and is the ratio of the yield point unit stress of the material (steel in this case) to the working unit stress as determined by the straight line theory.

The straight line method which assumes a uniform theoretical safety factor does not result in uniform factors of safety and section economy at critical sections of the frame under combined direct and bending stress. This is demonstrated by curves in Fig. 2, where the factor of safety for the straight line method is shown to vary from 2.44 to 3.84. Sections designed with needlessly large factors of safety produce higher stresses throughout the structure.

To better illustrate these points, there are presented herein comparative designs for a skewed segmental frame bridge. The bridge selected as an example has a 45 deg. 36 min. skew, a clear right span of 40.83 ft and has other dimensional properties as illustrated by Fig. 3, Fig. 4, and Table 1.

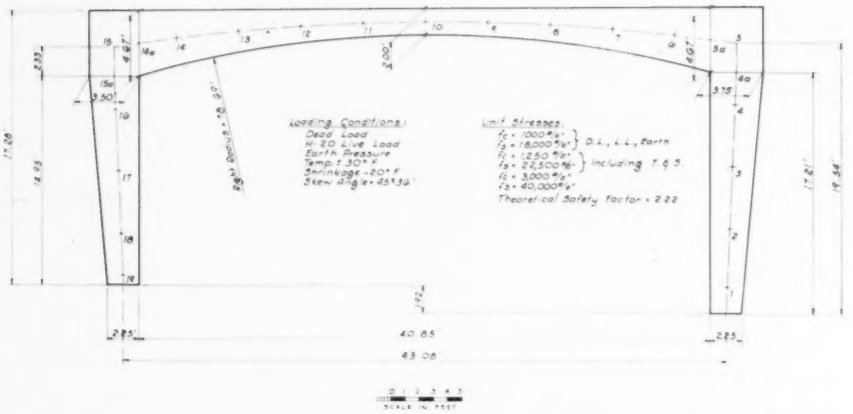


Fig. 3—Dimensions of concrete frames as determined by straight line theory

Based on the straight line theory, the section thickness t and reinforcing steel A_s are shown in columns C and D of Table 1. The theoretical safety factors are noted on Fig. 3.

Using the same shape, section size, and reinforcing steel areas, the factor of safety as computed by the plastic theory is listed in column E of Table 1. The actual factors of safety as determined by use of the plastic theory do not differ appreciably, except at the crown, from the theoretical safety factors obtained from the straight line theory. This phenomenon is evident from Fig. 2 which compares the ultimate and straight line methods and shows that in members with little thrust and with steel percentage below 1 percent the resistance to moment is practically identical by either method of analysis. In sections having low steel ratios the concrete is greatly understressed, as may be concluded from values in column F of Table I where the factor of safety of the same section fully reinforced is indicated. Although it is to be noted that a "balanced" design may be unnecessary or even impractical from a construction viewpoint, however, a step in this direction is desirable. It was therefore decided to reproportion the frame utilizing the greater strength actually available in the concrete in flexure. Table 1, column G shows approximate minimum thickness of critical sections possible had the concrete been fully reinforced. The shape of the frame based on balanced designs at the critical sections is shown in Fig. 4. However, it was decided to limit the reduction in thickness for several reasons—namely: (1) to restrict the steel placement to one layer in both the extrados and intrados for ease of concrete placement and maximum economy; and (2) to provide an intrados curvature which would present a desirable appearance of the ringstones on the portal face. The shape of this revised frame is shown in Fig. 4 and Fig. 5.

TABLE 1

	Section size based on straight line theory							Section size based on plastic theory				
	A	B	C	D	E	F	G	H	I	J	K	M
Point	Moment, ft kips	Thrust, kips	Thick, ft	Steel† required, sq in.	Factor of safety based on plastic theory	Factor of safety of fully reinf. section	Min. thickness plastic theory fully reinf. section	Moment, ft kips	Thrust, kips	Thick, ft	Steel† required, sq in.	Factor of safety based on plastic theory
H_R	0											
1	− 20.44	36.42	2.42	—				− 18.39	29.81	1.85	—	
2	− 63.99	33.99	2.78	.59				− 55.54	27.97	2.14	.99	
3	−109.95	31.16	3.17	1.45				− 92.79	25.82	2.41	1.92	
4	−155.34	27.96	3.56	2.08				−129.72	23.45	2.67	2.68	
4a	−177.69	26.28	3.75	2.34	2.34	9.32	2.17[1]	−151.99	22.21	2.83	3.06	2.29
5								−208.97	19.36			
5a	−224.14	12.73	4.67	2.76	2.35	12.50	2.23[1]	−207.69	11.07	3.58	3.44	2.29
6	−153.25	12.65	4.00	2.07				−130.29	11.07	3.00	2.53	
7	− 80.62*	13.48*	3.14	.98				− 50.09	10.41	2.31	1.09	
8	+ 76.31*	9.13*	2.50	+1.40				+ 50.00*	8.60*	1.87	+1.24	
9	+109.34*	10.30*	2.11	+2.63				+ 77.56*	9.71*	1.59	+2.77	
℄10	+121.16*	10.79*	2.00	−1.25	2.05*	4.63	1.42[1]	+ 72.36	10.92*	1.50	− .44	1.94*
				+3.54	3.04						+3.62	2.35
11	+115.79*	10.85*	2.11	+3.20				+ 83.27*	9.75*	1.59	+3.03	
12	+ 87.61*	9.15*	2.46	+1.75				+ 60.43*	8.60	1.87	+1.56	
13	− 58.18*	13.52*	3.12	.66				− 44.18	16.05	2.35	.74	
14	−124.08	12.63	3.96	1.66				−115.23	16.65	3.00	2.02	
14a	−197.48	12.71	4.67	2.31	2.34		2.23[2]	−185.78	16.67	3.58	2.88	2.30
15								−183.80	20.44			
15a	−152.10	25.67	3.50	2.11	2.32		2.17[2]	−131.62	21.34	2.50	3.14	2.34
16	−127.89	27.23	3.30	1.78				−108.54	22.20	2.34	2.62	
17	− 82.92	30.19	2.92	1.10				− 73.46	24.30	2.14	1.70	
18	− 37.45	32.80	2.55	.05				− 38.13	26.22	1.93	.59	
19	− 7.26	34.36	2.31	—				− 12.28	27.50	1.78	—	
H_L	0											

*Includes temperature variation.

†Negative sign in columns D & K indicates compressive reinforcement.

[1]Column G approximate minimum size—using straight line moment and thrust and *without* recalculating for reduction in moment based on minimum section and without compression steel.

[2]For symmetrical dimensioning (not governed by capacity).

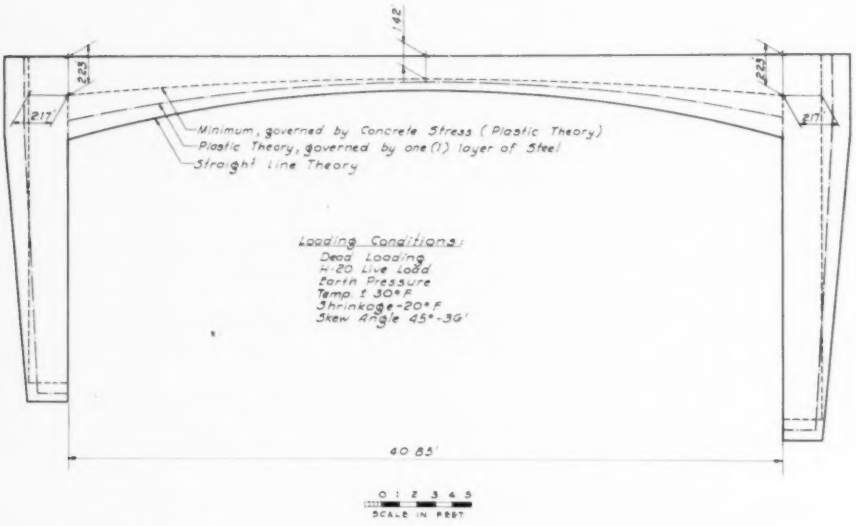


Fig. 4—Dimensions of concrete frames as determined by straight line and plastic theories

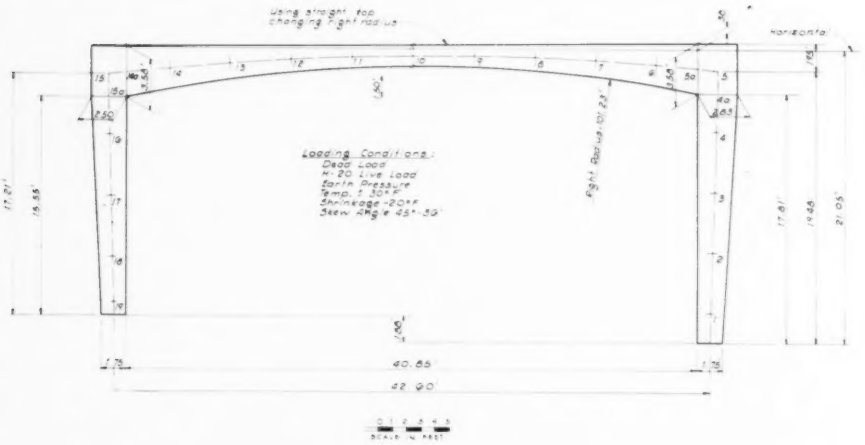


Fig. 5—Dimensions of concrete frame as determined by plastic theory

It is well to point out here that in some cases the full utilization of the minimum depth of section permitted by the plastic theory may result in a structure having excessive deflections.

Analysis of the frame using these revised sections disclosed that if 25 percent increased stress is allowed for combinations including temperature and shrinkage, the stresses from volume changes no longer controlled the section dimensions. The experienced designer, who has

gone through numerous contortions in an effort to minimize such stresses will fully realize the importance of this feature. The depth of sections and required steel areas are shown in Table 1. columns J and K; the factors of safety are indicated in column M, Table 1. The almost constant value of the factor of safety should be noted.

Under the straight line design the frame has been proportioned so as to require the introduction of considerable compression steel at the critical crown section. This proportioning is adopted to permit the reduction of dead load and to overcome the otherwise adverse distribution of moment between that at the crown and that at the springing line and to add to the general flexibility of the frame. The plastic theory shows that this compressive steel, if used in the tensile region instead of in the compressive region, would have increased the crown capacity by 18 percent, resulting in a stronger structure. Practical conditions, such as anchoring of ties, and the desirability of restricting plastic flow deformations, indicate the necessity for some reasonable amount of compression steel at the crown section; nevertheless, it is well to understand the truer picture of the member capacity and its limitations at this most critical section of the frame.

The results of the previously noted calculations were based on the assumption of cracked sections. Considering the dead load as applied prior to the application of live load and making allowances for plastic flow, $n = 30$ for dead load and $n = 10$ for all other loads, the revised frame would theoretically behave as a cracked section only in the area immediately adjacent to the crown. Design data substantiating this conclusion are shown in Table 2. This is of special importance in the interpretation of results from model tests.

The use of the frame of the more slender proportions would result in (1) economy of concrete in the frame and footings, (2) only small increases in the total reinforcing steel areas, (3) a decrease in the centering required for erection, and (4) a reduction in earth work quantities and wingwall heights. For frames founded on piles, the reduced total reactions would be of material economic value.

The slenderer frame was designed to be at least equivalent to the original frame under all design loads. Additional comparisons are made to obtain the relative strength under variations of loading not generally specified in order to determine the over all desirability of each frame. Table 3, indicating the factors of safety for various loading combinations, is presented for this purpose.

According to the provisions specified by AASHO-44* it is necessary that the following conditions be satisfied, assuming the use of inter-

*"Standard Specifications for Highway Bridges" adopted by The American Association of State Highway Officials,—Fourth Edition, 1944, p. 143.

TABLE 2—STRESSES ON UNCRACKED SECTIONS. SECTION SIZE BASED ON PLASTIC THEORY DESIGN

	Concrete section only assumed effective							Concrete and steel sections assumed effective, $n = 10$			Concrete and steel sects. assumed effective, $n = 30$ for dead load; $n = 10$ for other loads			
Point	Moment, ft kips	Thrust, kips	Concrete provided sq in.	Direct stress, psi	Sect. Modulus, in.^{3}	Bending stress, psi	Maximum tensile stress, psi.	Sect. modulus, in.^{3}	Bending stress, psi	Maximum tensile stress, psi	Sect. modulus, in.^{3}	Dead load bending stress, psi	Live load bending stress, psi	Maximum tensile stress, psi
H_R														
1	— 18.39	29.81	267.8	111	995	±221	110							
2	— 55.54	27.97	309.6	090	1330	500	410							
3	— 92.79	25.82	348.5	074	1685	660	586							
4	−129.72	23.45	385.9	061	2062	756	695							
4a	−151.99	22.21	406.1	054	2290	797	743	2820	±648	594	4000	330	179	455
5	−208.97	19.36	489.6	040	3340	750	710							
5a	−207.69	11.07	516.0	021	3700	673	652	4400	566	545				
6	−130.29	11.07	432.0	026	2590	605	579							
7	— 59.40*	10.41*	334.1	031	1550	460	429							
8	+ 50.00*	8.60*	270.7	032	1020	588	556							
9	+ 77.56*	9.71*	230.4	042	740	1260	1218	988	943*	901*	1470	306	487*	751*
℄10	+ 87.63*	10.13*	216.0	047	650	1620	1573	869	1210*	1163*	1300	436	559*	943*
11	+ 83.27*	9.75*	230.4	042	740	1350	1308	988	1010*	0968*	1470	348	495*	801*
12	+ 60.43*	8.60*	270.7	032	1020	710	678	1330	545*	513*				
13	— 52.82*	16.51*	339.8	049	1600	396	347							
14	−115.23	16.65	432.0	039	2590	535	496							
14a	−185.78	16.67	516.0	032	3700	603	571							
15	−183.80	20.44	468.0	044	3050	723	679							
15a	−131.62	21.34	360.0	059	1800	878	819							
16	−108.54	22.20	338.4	066	1590	818	752	1900	±685	619	2580	360	196	490
17	— 73.46	24.30	309.6	079	1330	663	584							
18	— 38.13	26.22	279.4	094	1085	421	327							
19	— 12.28	27.50	257.8	107	921	±160	53							
H_L														

*Includes temperature variation.

TABLE 3—FACTORS OF SAFETY FOR VARIOUS LOADING COMBINATIONS (Y = ZERO)

Loading combination			Sect. 4 a.		Sect. 5 a		Sect. 10		Sect. 14 a		Sect. 15 a	
Line	Equation		St. L.	Pl. Th.	St. L.	Pl. Th.	St. L.	Pl. Th.	St. L.	Pl. Th.	St. L.	Pl. Th.
A	(1b)	$D + L + I + E$	2.34	2.29	2.35	2.29	3.04	2.35	2.34	2.30	2.32	2.34
B	(5b)	$D + L + I + E + Y + T$	2.16	2.19	2.18	2.18	2.05	1.94	2.11	2.17	2.14	2.25
C	(3b) (4b)	$D + 2L + 2I + E + Y + T$	1.68	1.70	1.75	1.70	1.69	1.50	1.70	1.71	1.70	1.75
D	—	$D + L + I + E + W + Misc. + 2.4(Y + T)$	1.89	2.10	1.95	2.07	1.41	1.56	1.90	2.07	1.94	2.11

mediate grade reinforcing steel based on 18,000 psi working stress and 40,000 psi yield point value:

$$40{,}000 \text{ psi} \geqq X\,(D + L + I + E) \quad \ldots\ldots (1)$$

$$40{,}000 \text{ psi} \geqq Z\,(D + L + I + E + Y + T + W + M) \quad \ldots\ldots (2)$$

Where D = dead load
L = live load
E = earth pressure
T = temperature
W = wind pressure
M = miscellaneous loadings
I = impact
Y = footing movements

The terms within the parentheses represent the stresses derived from the summation of moments and thrusts acting on a section from the loadings noted.

Bearing in mind that at no section will there be incorporated sufficient steel to result in a balanced design, and using the specified steel stresses (since in the ultimate stress condition steel will govern); $X = 2.22$; $Z = 1.78$; Eq. (1) and (2), as they apply to rigid frames, may be rewritten as follows:

$$2.22\,(D + L + I + E) \lessapprox 40{,}000 \quad \ldots\ldots (1a)$$

$$1.78\,(D + L + I + E + Y + T) \lessapprox 40{,}000 \quad \ldots\ldots (2a)$$

since wind and miscellaneous stresses such as those from rib shortening and secondary deflections may be disregarded.

Whitney suggests for consideration the following formulas for use in conjunction with the plastic theory:

$$1.2\ (D + 2L + 2I + E + R + S + Y + T) \lessapprox U \quad \ldots\ldots (3)$$

$$1.2\ (D + E + R + S + Y + T + 2W) \lessapprox U \quad \ldots\ldots (4)$$

$$1.67\ (D + L + I + R + S + Y + T + \tfrac{1}{2}W) \lessapprox U \quad \ldots\ldots (5)$$

$$1.67\ (D + \tfrac{1}{2}L + \tfrac{1}{2}I + R + S + Y + T + W) \lessapprox U \quad \ldots\ldots (6)$$

Where R = rib shortening
S = secondary bending effect caused by deflection

*Includes temperature variation.

The terms within the parentheses represent the working thrust and U the ultimate thrust capacity, each acting at the same eccentricity. The value of U is dependent on the yield point value of the reinforcement and the area of steel provided. For unit steel area, $U = 40{,}000$ lb.

For the case under consideration, Eq. 3 to 5 may be rewritten as follows:

$$1.2\ (D + 2L + 2I + E + Y + T) \lessapprox 40{,}000 \quad \text{(3a) (4a)}$$
$$1.67\ (D + L + I + E + Y + T) \lessapprox 40{,}000 \quad \text{(5a)}$$

Eq. 3 to 6 are based on a working stress of 20,000 psi for intermediate grade steel. Assuming a working stress of 18,000 psi as is often the practice in rigid frame bridge design, Eq. (3a), (4a) and (5a) may be rewritten as follows:

$$1.33\ (D + 2L + 2I + E + Y + T) \lessapprox 40{,}000 \quad \text{(3b) (4b)}$$
$$1.85\ (D + L + I + E + Y + T) \lessapprox 40{,}000 \quad \text{(5b)}$$

The lever arm of the steel in the plastic theory, where the percentage of steel is relatively low (which is usually the case in frame design), is somewhat greater than the corresponding value in the straight line theory. To compensate for this, it is recommended that the value of 2.22 in Eq. (1a) be changed to 2.30, and the value of 1.78 in Eq. (2a) be changed to 1.85 and restated as follows:

$$2.30\ (D + L + I + E) \lessapprox 40{,}000 \quad \text{(1b)}$$
$$1.85\ (D + L + I + E + Y + T) \lessapprox 40{,}000 \quad \text{(2b)}$$

Eq. (2b) and (5b) are identical. Eq. (3b) (4b) is set up to prevent failure due to overload, equation (1b) is to secure an adequate proportioning when the effect of temperature and shrinkage is of minor importance, such as would be obtained with frame of small or zero skew angle, and because of the uncertainty of the results of present skew frame analysis.

Due to the small ratio of live load to dead load obtained in highway rigid frame bridges, Eq. (3b) (4b) will, in most cases, not govern the design. See Table 3, line C, which indicates the factor of safety for this particular bridge as being well above 1.33.

Therefore, for highway rigid frames only the following conditions need be satisfied generally:

$$2.30\ (D + L + I + E) \lessapprox 40{,}000 \quad \text{(1b)}$$
$$1.85\ (D + L + I + E + Y + T) \lessapprox 40{,}000 \quad \text{(5b)}$$

If 20,000 psi is to be applied, (1b) and (5b) are to be rewritten as follows:

$$2.07\ (D + L + I + E) \lessapprox 40{,}000 \quad \text{(1c)}$$
$$1.67\ (D + L + I + E + Y + T) \lessapprox 40{,}000 \quad \text{(5c)}$$

Eq. (1b) and (5b) were applied in the case of the two frames under consideration, Eq. (1b) governed at all sections except at the crown.

TABLE 4—TRANSVERSE (TORSIONAL) STEEL REQUIREMENTS (SQ IN. PER FT.)

	Structure proportioned by straight line theory	Structure proportioned by plastic theory
In deck of frame	1.150	0.975
In leg of frame	0.292	0.218

Comparison of the two frames as noted in Table 3, indicates that the drop in factors of safety is practically constant at all critical points for an increase of live load of 100 percent, and examination of lines B and D show that the loss in factor of safety of the slenderer frame under temperature variation is less than 50 percent of the loss in factor of safety in the original frame. In every case the slenderer frame is equivalent or superior to the original frame under design load and considerably tougher and stronger for miscellaneous loads not considered in the design and under loads subject to the least accuracy in analysis.

Another condition not included in the above design concerns possible movements of the footings. Assuming a ¼-in. spread at each footing, the factor of safety drops 40 percent in the original frame, while dropping only 16 percent in the slenderer frame.

The maximum right span dead load deflection for the frame shown in Fig. 3 was computed as 0.12 in. The deflection of the revised section was 0.20 in. Neither deflection is of sufficient magnitude to require revision in the frame proportions.

Transverse (torsional) steel requirements in the deck and leg were determined for the frame proportioned by the straight line theory and that by the plastic theory. Results are shown in Table 4. In each case the required areas per ft were less in the slenderer frame. This is due in part to the reduced dead load and volume change reactions and also due to the reduced stiffness of the frame members in torsion.

CONCLUSIONS

Adoption of the plastic theory in proportioning rigid concrete frames would lead to more economical proportions, yet yielding a structure capable of sustaining equal or greater loads. At the same time the effects of volume changes and yielding of supports are of smaller consequence on such frames.

It is recognized that further substantiation of the plastic theory as it may apply to concrete frames is desirable, and the writer, as a member of Committee 314—Rigid Frame Bridges, has proposed tests of small and full size three dimensional models of right and skewed span rigid frame concrete bridges, proportioned by both the straight line and plastic theory, as part of its work.

In September **Letters from Readers** replaced **Job Problems and Practice,** with broader scope, more flexible form, and we hope more adaptability to the thoughtful, busy reader's use, as a forum for opinions, experiences and inquiries within the field of ACI activity. A letter does not need the studied treatment of a major paper to present valuable ideas for ready access to kindred minds.

Letters from Readers*

BY WAY OF SYNOPSIS

F. L. FITZPATRICK says that effective application of research discoveries to construction problems requires the constructor or manufacturer to maintain his own experimental testing program.

An international conference on concrete and reinforced concrete is proposed by MICHEL BAKHOUM.

Research Made Practical—for Pipe and Products Makers
(LR 45-11)

F. L. FITZPATRICK, general manager of Rocla, Ltd., Melbourne, Australia, acknowledges the value of research as applied to concrete products manufacturer.

It has been truly said that the world suffers less from the lack of research than from the failure to apply, in practice, the results of research already done. In the field of concrete pipe and products manufacture, this statement holds good to a very high degree. These comments are directed particularly toward the concrete pipe industry, but are true, I think, in general of people using concrete in all fields.

Published matter from such research bodies as the Portland Cement Association and the Bureau of Reclamation in the United States, England's Building Research Station and many other quarters, provides a basis for action in many directions. In the U.S.A. as in many other countries, the pipe industry has not always been quick to appreciate

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and apply the new knowledge made available to it. Consequently, concrete manufacturing procedures have lagged behind the progress in concrete knowledge.

It is sometimes said by those engaged in industry that much of the published research is "too academic" and of little practical use. This view is too often founded on a failure to appreciate the full significance of a new development that may appear "academic". It is founded, too, on a wrong conception of the functions of the research bodies.

It is our view, in my company, that we can expect assistance from outside researchers only on the fundamentals of the materials we work with. Our own problem lies first in properly evaluating the significance to ourselves of the published information, and second in planning our own experimental work, to weave into our procedures the improvements made possible by the new information. Our plan therefore is this:

(1) To arrange that competent people study all the available literature, with our own problems in their minds.

(2) To initiate our own series of trials and experiments, applying the published material to our own problems and conditions.

(3) To incorporate in our working methods those improvements found desirable.

There has to be a link between the research institution and the working plant. That link should be the research or experimental section of the manufacturer himself, operating on the lines mentioned. We ourselves have found that great practical benefits, in the way of better products and lower costs, are so achieved. We have found, too, that the published work of research bodies has very great value in our work. Without it, we could not have achieved what has been done.

Our general plan is this:—

(1) We have what we call the "chemical research section". This section is concerned with the concrete itself, with admixtures, coatings, treatments—with problems of corrosion, physical properties and the like.

(2) The chemical research section is equipped with analytical apparatus, briquette-machines, storage tanks for aggressive solutions, oven and furnace, tension and compression testing machines, strain-measuring equipment, microscopes and other incidental apparatus. It explores the possibilities inherent in any new ideas, with respect to our work, and solves all chemical problems arising in the manufacture and use of our products.

(3) The engineering section is responsible for studying the matters of design, manufacturing processes and manufacturing equip-

ment, and for experimenting upon and initiating new manufacturing procedures, whether based on the work of the chemical research section or not.

(4) There is a planned system of studying and indexing all relevant published information available.

Examples of the practical working of the system can be cited. A problem arose where a pour of several feet had to be made in a thin concrete member. Due to horizontal projections of the member, trouble was experienced due to slumping and consequent cracking. Using the information gleaned and recorded from the writings of investigators on air entrainment, the procedure was evolved of using air-entraining concrete, closing the mold after filling with concrete, and then heating it. The slight swelling of the included air offset the slumping tendency—the problem was solved swiftly and economically.

The need arose for a pozzolanic material in our work. The published work of German and U.S. investigators (including Professor Davis at the University of California) was made the basis for work by our own chemical research section upon locally available materials. A first-class pozzolan was developed, the necessary machinery installed, and our whole business was strengthened and its usefulness enlarged.

Similarly, in the fields of prestressing, in the extensive manufacture of reinforced concrete pipes (not prestressed) for high pressure water-supply work, in developing protective coatings, in improving centrifugal pipe manufacturing procedure, the published work of many researchers (often not at first sight relevant to the subject), has been of profound importance to the progress of our work. We are greatly indebted to the painstaking work, and lucid expositions thereof, of the men whose findings appear in such publications as the JOURNAL of the American Concrete Institute, and those of the (American) Portland Cement Association, and the universities and other institutions, especially of America and England.

In our view, there remains much to be done, on the part of engineers, manufacturers, builders, and those doing the construction work, toward absorbing and applying the mass of information available to them. It is their responsibility to make the research work "practical". The research institutions cannot be expected to present their work in a form applicable to a million-and-one differing local circumstances. They provide the background information—it is we who must work out its significance and application to our own particular work and problems. It pays us well to do so, too.

An International Conference On Concrete (LR 45-12)

is proposed by MICHEL BAKHOUM, lecturer of the Faculty of Engineering at Giza, Egypt.

May I suggest that the Institute make inquiry of members and others whether it is desirable to have an "International Conference on Concrete and Reinforced Concrete" in the near future. As far as the writer knows, the last conference was the one in Belgium in 1930. It succeeded in giving a clear idea of the state of knowledge of concrete at that time. Since then, however, numerous gains have been made: new theories of design have been developed and even introduced into specifications in some countries; extensive studies have been made of the properties of concrete and of the behavior of reinforced concrete elements under different stresses; new materials, equipment and methods of construction have come into use and a great variety of important concrete structures have been built. Another international conference would help crystallize the advancements and give impetus to more progress.

I suggest also that in such a conference no differentiation should be made between allies and former enemy countries during the war. This is not only to benefit from the contributions of such countries as Germany, Austria, Italy and Japan but mainly because it is time now to heal the scars of the war through mutual cooperation.

Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

Design and control of pumice concrete mixes

WALTER K. WAGNER, *Rock Products*, V. 51, No. 9, Sept. 1948, p. 125. Reviewed by T. M. KELLY

The problems involved in the control of pumice concrete are discussed and a method of mix design is presented. Control of gradation is of great importance since the absorption and specific gravity of the different sizes vary over a wide range. Pumice concretes tend to bleed and are benefited by certain admixtures. Pumice concrete without admixture contains normally 3.5 to 6.5 percent air. The mix design data, all refering to pumice from one source, were obtained from 85 mixes and five hundred 6 x 12-in. cylinders. Mix design is based upon the fineness modulus of total solids (by absolute volume). The proper value of fineness modulus of total solids is determined from one or more mixes and is then kept constant for all other mixes from lean to rich. Some data are presented in support of the use of pumice concrete for structural purposes.

Structures of prestressed concrete erected to contain or retain liquids (Ouvrages en beton precontraint destines a contenir ou a retenir des liquides)

E. FREYSSINET, International Association Bridge and Structural Engineers *Preliminary Publication* 3rd Congress, 1948, pp. 343-360 APPLIED MECHANICS REVIEWS Sept. 1948 (Durelli)

Some of the most revolutionary techniques in prestressed reinforced concrete construction are reviewed in this paper. The author constructs pipes of concrete having 17,000 psi compressive strength, and makes them by a continuous process using heat, vibration and pressure, and a very elaborate mold. The bars are prestressed longitudinally and transversely. Such pipes of 3, 4, and 5-in. diameters are in use in Algeria and are able to withstand pressures of 80 to 250 psi, and remain completely watertight. The author describes briefly the theoretical background of this new type of construction and also gives some information on methods used to prestress water and wine reservoirs, and large dams. The paper is general in character, giving few details but it shows extraordinary possibilities of development in reinforced concrete.

What causes defective concrete?

J. A. SLEGTEN, *Rock Products*, V. 51, No. 10, Oct. 1948, p. 95 Reviewed by T. M. KELLY

Excessive additions of gypsum are suspected as the source of defective concrete. The compressive to tensile strength ratios of cements have gradually increased. Com-

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pressive strengths increase up to a 7 percent addition of gypsum while tensile strengths fall off when gypsum exceeds 5 percent. The author believes that the agglomeration of calcium sulfoaluminate within the pores continues until the pores are filled, and expansion and decreased tensile strength result. Compressive strength increases to a point, but as the gypsum addition is increased, ultimate disintegration of the concrete or mortar will result. The possibility is also suggested that, even with small percentage additions of gypsum, the same danger exists after a period of months or years. Laboratory investigations to study the effect of gypsum are recommended.

Large span bridges of prestressed concrete in Belgium (Ponts de grande portee en beton precontraint realises en Belgique)

A. Paduart, International Association Bridge and Structural Engineers, *Preliminary Publication* 3rd Congress, 1948, pp. 325-332

Applied Mechanics Reviews Sept. 1948 (Durelli)

Reinforced prestressed concrete techniques permit light and efficient structures. However, some questions concerning its behavior limit its wide use. The author describes an experimental beam 90 ft long and 2.7 ft high, with an arch of 1 ft, which was built to check deformations and ease of prestressing. Straight tie rods 1⅝ in. in diameter located in the open side of the inverted U-section, were used for prestressing. The concrete work was finished in a day. Static and dynamic tests are compared with theoretical calculations. A pedestrian bridge 140 ft long was built on the basis of this experience.

Studies of the Mattson shot classifier

Raymond L. Blaine and Harold J. Valis, *Journal of Research* National Bureau of Standards, V. 41, Nov. 1948

Reviewed by C. C. Fishburn

The size and distribution of peening and cleaning shot and sand particles of various shape characteristics were determined by use of a new apparatus and with a microscope and by micro-weighings. Tests indicated that the Mattson apparatus enabled a rapid visual evaluation to be made of size and distribution of shot and sand particles. A more precise evaluation can be made with the apparatus by determining the actual size distribution of the sand or other particles. This apparatus tends to determine the smallest dimension of particles that are irregular in shape and, when used with closely sized sieve fractions, makes possible an evaluation of the shape characteristics of the particles.

Workability properties of fresh concrete

P. Hallström, *Betong*, (Stockholm), V. 33, No. 3, 1948, pp. 125-138.

Reviewed by Eivind Hognestad

The most important properties of the fresh concrete affecting its ability to form a homogeneous hardened concrete are discussed in considerable detail in terms of: (1) stability, (2) cohesion, (3) fluidity, and (4) mobility.

The paper describes how the grading of the aggregates, the cement content, and the flocculation of the cement affect the above properties. Ways and means of increasing the flocculation, such as increasing the fineness of the cement, "activation," heating, etc. are discussed. The effect of accelerators, dispersing agents, and air-entraining agents is mentioned.

The difference between fluidity and mobility is emphasized. It is not sufficient to demand a certain fluidity or consistency of concrete for a given structure. A suitable mobility should also be specified.

When the vibrator contacts the reinforcing steel

J. R. Shank, Ohio State University, *Engineering Experiment Station News* V. 20, No. 4, Oct. 1948, pp. 7, 8

Effect of vibrator contacting reinforcement in partially hardened concrete was studied. Reinforcement in pull-out specimens was vibrated at ages from 2 to 48 hours; specimens were then cured and tested for bond. Results shown in Fig. 1 indicate that vibration up to 8-hour age improved the bond, but from 8 to 24 hours it was detrimental, particularly so in the 8 to 16-hour period. After 24 hours no bad effect developed.

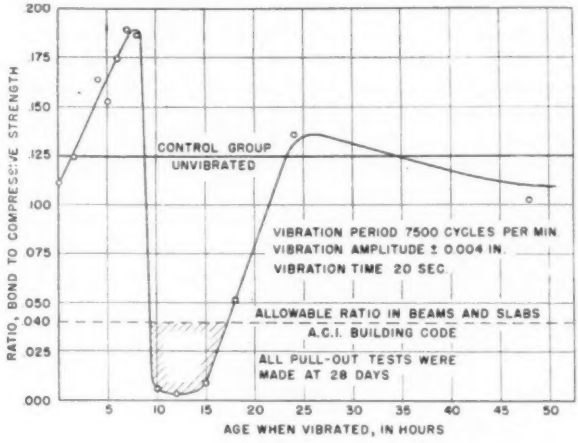


Fig. 1

Expansive characteristics of hydrated limes and the development of an autoclave test for soundness

Lansing S. Wells, Walter F. Clarke, and Ernest M. Levin, *Journal of Research* National Bureau of Standards, V. 41, Sept. 1948 Reviewed by C. C. Fishburn

A technique for preparing, curing, and autoclaving 1 x 1 x 10-in. cement-lime bars was developed, and the expansive characteristics of 80 commercial hydrated limes were determined. On the basis of chemical analysis and percentage of unhydrated oxide, the hydrated limes were classified into four series: high-calcium, regularly hydrated dolomitic, highly hydrated dolomitic, and magnesian. Data on the expansions of cement-lime bars showed that bars prepared with the regularly hydrated dolomitic limes, which had the highest percentages of unhydrated oxides, had the highest percentages of expansion. The high-calcium limes, characterized, in general, by the lowest percentages of unhydrated oxides, gave the lowest percentages of expansion. Most of the highly hydrated dolomitic limes had percentages of unhydrated oxide and expansion that were comparable to those of the high-calcium limes. An increase in the proportion of lime in the cement-lime bars was attended by an increase in expansion. The effect of 17 different portland cements on the expansion of cement-lime bars showed that the expansion values for a particular lime tended to increase as the expansion value of the constituent cement increased. Arbitrarily subtracting the expansion of the neat cement from the total expansion gave the most uniform result for the "net" expansion of the lime. With an autoclave specially modified for ascertaining the behavior of cement-lime bars during the course of autoclaving, it was found that only a slight amount of the total expansion occurred before a temperature of 150 C was reached, but above 150 C a rapid expansion rate was noted, which in turn tapered

off before a temperature of 216 C was reached. Retarding the rate of heating resulted in a decrease in the total expansion. Finally, from criteria set forth for a procedure for determining the soundness of hydrated limes, a test is proposed with a suggested limit of expansion of 1.0 percent.

Design of beams on elastic foundations by the method of Biezeno (Zur Berechnung von Traegern auf elastischer Unterlage nach dem Verfahren von Biezeno)

W. MUDRAK, GRAZ, *Oesterreichische Bauzeitschrift* (Vienna), V. 3, No. 8-9, Aug.-Sept. 1948, pp. 125-131 Reviewed by RUDOLPH FISCHL

The method of Biezeno first considers a generally loaded beam on an elastic foundation to be infinitely stiff and loaded at the center with the resultant of the vertical forces and a couple. The difference between the straight-line soil pressure under the fictitious beam and the real soil pressure must be in equilibrium within itself.

With help of a rapidly converging series, an approximation to the actual soil pressure can be made so close that the derivation of the internal stresses of the beam is as good as exact for practical purposes.

The first members of the series will indicate whether the assumed dimensions of the beam are applicable or not. This is an advantage over the classical method using a beam of infinite length where one finds only at the end of the whole calculation whether the system corresponds to the original assumption.

An example interprets the application of the method. Seven illustrations and five tables, two for the practical use and three showing the application of the example, are included.

Introduction to Highway Engineering

JOHN H. BATEMAN, Fifth Edition, John Wiley & Sons, Inc., New York, 1948, $5.50 Reviewed by A. A. ANDERSON

In the preface the author states that the text book was written as an introductory course in highway engineering. To a large extent, the author has fulfilled that aim and covered each phase of the subject. Emphasis is on rural highway engineering practices, although general street design is also discussed.

Good bibliographies are listed at the end of each chapter. With this book and outside reading from the references given the student can gain a thorough grasp of highway engineering.

Four chapters outline the general procedure in working with subgrade soils, drainage, grading and general features of highway design. Seven chapters cover design and construction of flexible type surfaces together with bituminous surface courses.

The theories of design of concrete pavements and bases and block pavements are given in three chapters. The author has outlined the history of the structural design of concrete pavement from its inception in the "older" formula through Westergaard's analysis and the modifications suggested by E. F. Kelley and R. D. Bradbury.

Maintenance of all types of roads is discussed together with the organization and administration of maintenance departments. As highway engineering practices involve more than construction and maintenance, a chapter is included on surveys, plans and specifications.

The final chapters cover highway planning and economics. These cover the essential items and will give the student a good background for analyzing highway economics, either for individual projects or the entire system.

The text book is liberally illustrated and contains many pertinent charts and tables. It is well written and should give each student an excellent background for his further work.

Curing, a problem in thermodynamics (for the products manufacturer)

GEORGE A. MANSFIELD, *Rock Products*, V. 51, No. 8, Aug. 1948, p. 212 Reviewed by T. M. KELLY

Of interest to concrete products manufacturers, this is an enlightening discussion of the thermodynamic principles involved in high temperature atmospheric pressure curing of concrete. The author stresses the necessity for maintaining sufficient moisture in contact with the cement during the entire curing cycle. Otherwise, a brittle product results in which the cement has been damaged by the high temperature and lack of moisture. Caution is suggested if a curing system is selected wherein the products of combustion become a part of the kiln atmosphere. High concentration of carbon dioxide may cause carbonation of the concrete. If this occurs before the cement has hydrated it is detrimental; after hydration it is beneficial.

Boiler types, design of steam lines, and arrangement of nozzles are discussed. Nozzles at the ceiling are favored on the basis that the steam, being heavier than air, settles over the concrete units.

The curing cycle, consisting of the pre-setting period, the steaming period, the soaking period and the drying period, is discussed in detail. The pre-setting period (varying between 1 and 2½ hours) is necessary to allow the units to attain initial normal shrinkage before being exposed to the steam. Otherwise shrinkage cracks develop.

The author comments that the clause on absorption and moisture content in the standard specifications for concrete masonry units penalizes the quality producer.

Control of portland cement raw mixtures

E. J. SPOHN, *Rock Products*, V. 51, No. 8, Aug. 1948, p. 160 Reviewed by T. M. KELLY

The author comments on the conventional practice of controlling the raw mix by determination of the carbonate content. He discusses rapid methods of determination of carbonate content, noting that their use is limited by the fact that the carbonate requirement is not always exactly known and varies with the acid component and with *MgO*. *MgO* is determined as a basic component by the rapid methods, whereas in the clinker it is uncombined. Calculation of the exact carbonate requirement from the complete analysis of the clinker is credited with contributing substantially to our understanding of cement clinker, but objections to the use of this method for routine control are cited. These result primarily from the fact that appreciable variation in the raw material requires frequent complete analyses which are necessarily slow and result in high costs. The author presents a method of determining directly and quickly the amount of deviation from the lime requirement. The method is based on the determination of the free lime in the clinker. Tests are run on a sample which is drawn from the raw mix right after the proportioning point and which is burned under well-defined conditions in a laboratory furnace. The "conditions" are fixed by a preceding series of comparative tests on the "technical "clinker. A practical application of the method to a specific dry process plant is described.

Strength of large concrete pipe

W. R. MASON, *Proceedings*, 1948 Northwest Conference on Road Building, pp. 62-82 AUTHOR'S SUMMARY

The increasing use of large reinforced concrete culvert pipe has necessitated laboratory tests by the two recognized A. S. T. M. methods, the three-edge and the sand bearing. Currently accepted ratio of the required load carrying capacity by the three-edge test to the load required when testing by sand bearing is 2:3, regardless of size.

In August, 1947, the Washington State Highway Dept. requested the structural research laboratory of the civil engineering department, University of Washington,

to make actual tests by the two methods on the larger diameter culvert pipes (36, 48, 60, and 72-in. diameter) to determine the validity of the 1.0 to 1.5 ratio for these sizes. These tests indicate that this ratio holds for the 0.01-in. crack loadings for these larger diameters, but that for the ultimate loading it fluctuates between 1.0 to 1.5 for the 36-in. diameter and 1.0 to 2.0 for the 72-in. diameter pipes (not a linear relationship). The larger pipes are not as rigid as has been surmised; the 72-in. diameter pipe deflected vertically 3 in. and a corresponding amount horizontally, with the largest crack being on the average only 0.06 in. wide. This indicates that the larger pipes located in actual fills deflect sufficiently to enable the passive pressure of the soil on the vertical or horizontal projection of the pipe to give added load carrying capacity. Results on the larger culvert pipes are given in the body of this paper.

Concrete with aggregates of rubble of bricks, sandstones and debris (Ueber Ziegelsplittbeton, Sandsteinbeton und Truemmerschuttbeton)

O. Graf, *Mitteilungen der Deutschen Studiengesellschaft fuer Truemmerverwertung* (Hamburg), No. 2, 3, 4, Jan., Feb., March 1948 Reviewed by Rudolph Fischl

The German Research Society for Utilization of Debris is a growing organization which aims to unite the German building industry in the endeavor to provide high quality reconstruction. The technical and economic conditions expressing this aim have been published monthly. This paper by Professor Graf presents the basic technical knowledge concerning quality, preparation, shrinkage, and application of concrete using rubble of bricks and other debris as aggregates.

Brick rubble is suitable for cast-in-place concrete walls, pre-cast blocks and floor tiles, and for reinforced concrete only where strength is not important and lightweight is necessary. Special care has to be given to grading, and no fine aggregate must be added.

Test results show that a concrete with a minimum strength of 285 psi, containing brick rubble as the only aggregate, can be produced with 210 lb normal grade cement per cu yd or 152 lb per cu yd better grade cement or 118 lb high grade cement. Maximum weight of the cured concrete is 81 lb per cu ft.

This concrete is suitable for solid concrete blocks. The thermal insulating value of a 10-in. concrete-block wall is equal to that of a 15-in. normal brick wall. These blocks can be used for walls of two-story houses only. For cast-in-place walls of four-story houses, for hollow concrete blocks, and for tiles in joist floors a concrete with a strength of 710 psi to 1140 psi is required which can be produced by using 250 to 420 lb per cu yd of better grade cement or 300 lb of high grade cement.

Shrinkage is greater than in normal concrete but less than in other lightweight concrete.

The author does not recommend joists or girders to be built with this kind of concrete. Tests of its use in roof tiles are still going on.

Special care has to be taken in preparing the rubble to separate material with high content of gypsum. Tests with concrete containing 1.5 percent gypsum showed large cracks at the age of 7 days, and increased so much in length that it seems to be necessary to limit the amount of gypsum to 1 percent. Equipment for processing the aggregate is discussed.

Symposium on concrete floors

Cement och Betong, (Malmö, Sweden), V. 23, No. 3, 1948, pp. 83-113 Reviewed by Eivind Hognestad

Proportioning of mortar for wearing surfaces on concrete floors

V. Bahrner

An investigation is reported regarding consistency of cement mortars with five very different, typical sand gradations. Proportioning diagrams are developed with special

emphasis on those consistencies which were found to be suitable for finishing by hand and vibration respectively. For vibration 8 to 12 degrees Vebe are recommended. This mortar is so dry that it will not stick to the hands when a ball is formed.

Proposed classification of concrete floors
V. Bahrner

Concrete floors are classified with regard to various intensities of mechanical wear and attack by acids. On the basis of this classification, sand gradations, water content and cement content are recommended for wearing surfaces subject to various types of service.

Attack of chemicals on concrete floors
P. S. Hakanson

A brief consideration of the effect of various chemicals is presented. The use of admixtures can generally not be recommended. Proper proportioning, workmanship and curing are emphasized as the only rational basis for manufacture of a concrete surface which is able to resist a mild chemical attack. Unprotected concrete should not be used when strong chemical attacks are present.

Formation of dust on concrete floors
K. Hellberg

This article is based on a survey of inspections and interviews. The causes of dust formation are considered as being the laitance layer of the surface or the normal wear of the surface. Various admixtures and surface treatments are discussed. Treatment of the surface with oils or similar materials is recommended. Special recommendation is given to coal-tar pitch with benzol as a solvent when a cheap treatment is desirable.

Mechanical finishing of concrete surfaces
L. Forssblad

A brief review of the modern American equipment for surface finish, such as "trowels" and "power floats," is presented. AB Vibro-Verken, Stockholm has begun manufacture of similar equipment.

Building methods and building costs in Stockholm 1883-1939

H. J. Danielsson and M. Jacobsson, Swedish State Committee of Building Research, *Bulletin* No. 11, 1948. 100 pp. Reviewed by Eivind Hognestad

The bulk of Stockholm's buildings have been erected since 1880 when the large-scale migration to the capital began. Prior to that year the population of the city amounted to scarcely 170,000. In 1939, 580,000 people were living in the administrative part of Stockholm alone, the population thus having more than trebled in 60 years.

The object of the present study of building methods and costs during the period when most of the city was built, was to bring out facts suitable to serve as a basis in the frequently recurring discussions on the building costs of dwelling houses. The material has been condensed with due consideration to the following circumstances: (1) the technical development in the art of building; (2) the variation in building costs during different periods; and (3) the scale of dwelling house production in Stockholm. Type-houses have been selected which as closely as possible represent the most common building methods and equipment of their times. The considerations reported are based on houses which have really been built.

The structural materials have been developed considerably between 1883 and 1939. In the 1880's, builders were largely restricted to the use of bricks and timber. Steel and concrete later became increasingly dominant as materials for the supporting parts of houses. These materials also have allowed the use of light sawdust bricks and light-

weight concrete blocks, which had entirely ousted the heavy brick by the end of the 1930's. Around 1930, the wooden joist floor had to give way for the beam and girder concrete floor, which, finished with modern insulation materials, has far superior fireproofing and insulating capacities. The new building materials together with the increasing attention to the science of strength of materials have also substantially reduced the weight of buildings per unit floor area.

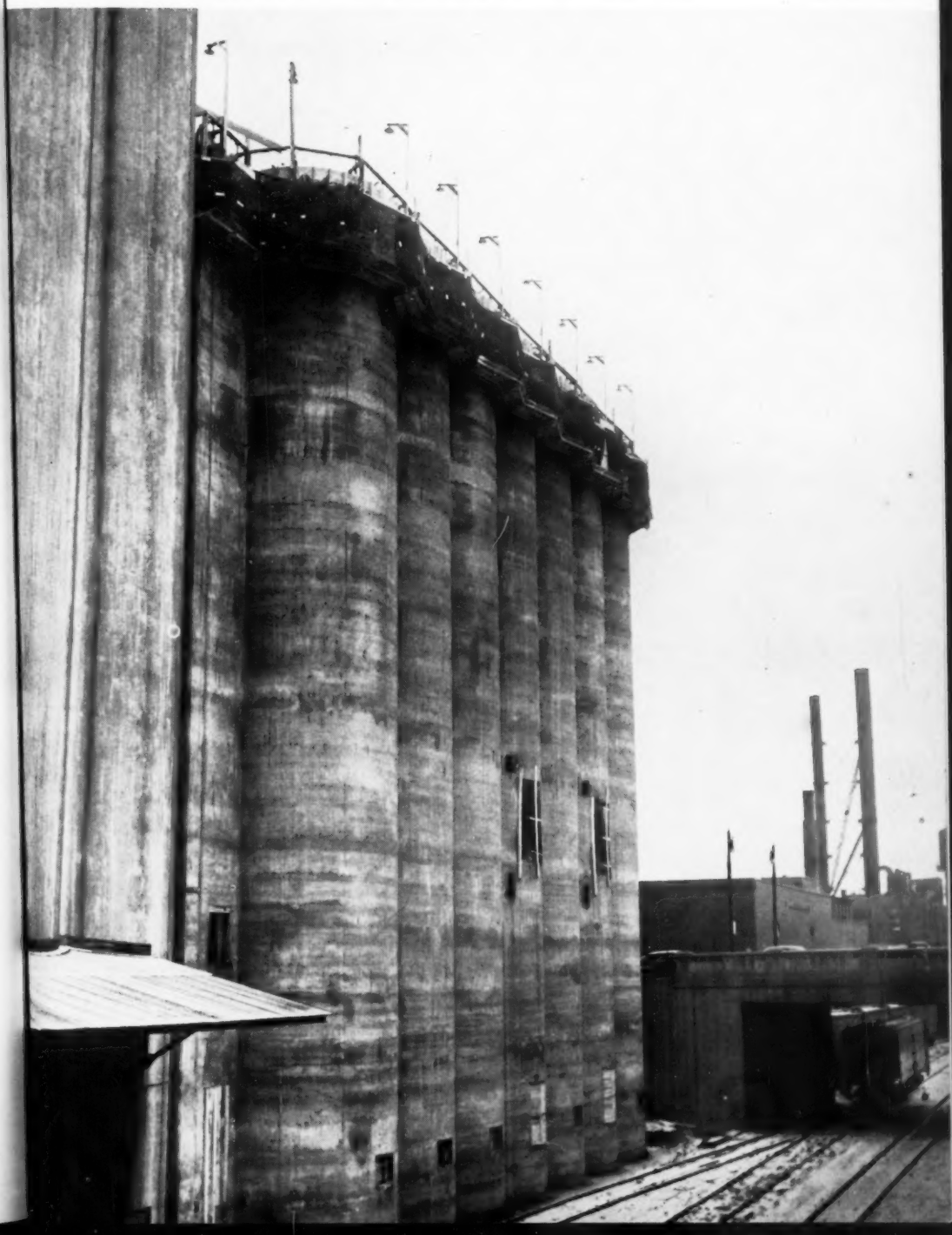
A large number of new lining materials have come into use in this century, such as tiles and marble, stainless steel sheeting, linoleum and parquet flooring, the latter being a normal feature of small apartments in Stockholm in 1939. Kitchen equipment, sanitary fittings, the manufacture of doors, windows, cupboards, etc. have been improved considerably.

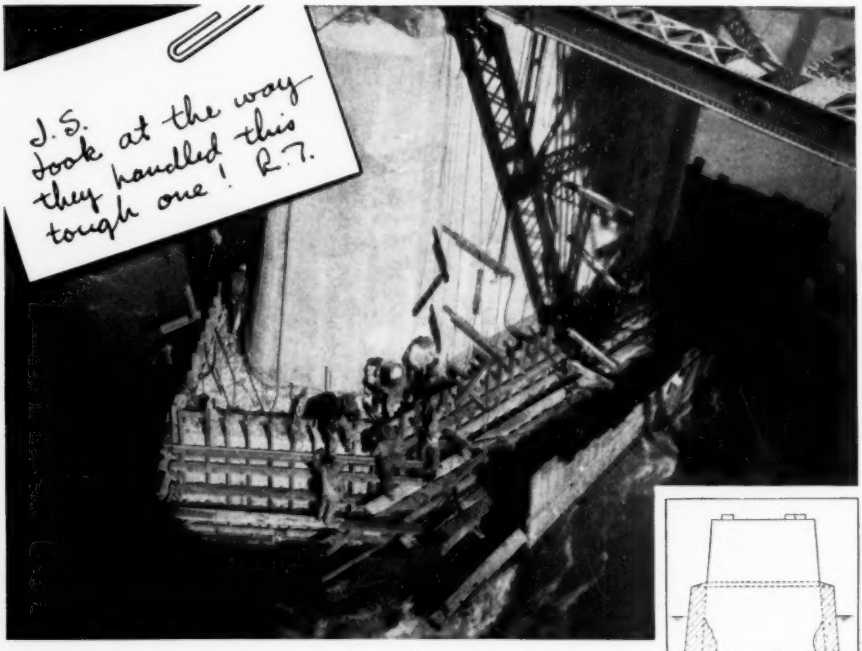
The rise in building costs has been studied with due regard to (1) price developments; (2) revised methods of production and (3) altered standards. An analysis of rising costs indicates that wage increases have contributed more than anything else. The wages have risen so much faster than the other factors in the building industry that the share in wages of the total building cost has steadily increased, despite the fact that the labor contributions to the completed house have diminished due to rationalization.

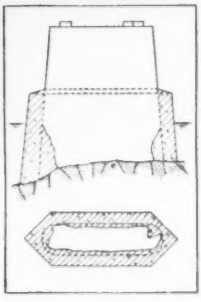
During the period studied, considerable qualitative improvements have entered the housing field. It is found that 40 to 50 percent of the price paid for a house in 1939 may be attributed to "modern conveniences" introduced since 1883. To build a completely modern house in 1939 cost about the same sum as it would have cost, with the prices of 1939, to build a house with 1883's conveniences using the production methods of 1883. Thus it can be said that the so-called rationalization has provided an improvement in standards virtually gratis.

ACI News Letter

Vol. 20 No. 5 JOURNAL of the AMERICAN CONCRETE INSTITUTE January 1949







PREPAKT REPORTS ON A . . .

TOUGH JOB, WELL DONE

Here is a case where the flexibility of PREPAKT methods and characteristics of INTRUSION-PREPAKT materials made possible an exceptional bridge pier restoration.

The small sectional sketch shows the extent of deterioration and the amount of repair.

By building a shield to divert the swift current, the forms were placed, the aggregate deposited, and the whole solidified by INTRUSION methods without a dewatering operation or the use of a cofferdam.

The ability of PREPAKT concrete to withstand freezing-thawing action and penetration of moisture insures a long and maintenance-free life to this pier structure.

THE CONCRETE WITH PREPAKT EXTREME DURABILITY

INTRUSION-PREPAKT, INC. THE PREPAKT CONCRETE CO.

CHICAGO · TORONTO CLEVELAND 14, OHIO PHILADELPHIA

ACI News Letter Contents

January 1949

On the cover—construction of monolithic reinforced concrete elevators at H. W. Rickel and Co., Maltsters, Detroit. Sliding steel-forms enabled the 120-ft total height of the $275,000 structure to be reached in 13 days. Contractor—Thomas Alburrell, Inc., Indianapolis, Ind.

Detroit Free Press photo

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A digest preview, 45th annual ACI convention Hotel Statler New York City*

Feb. 23-25, 1949

WEDNESDAY, FEBRUARY 23

9 a.m.—Foyer Grand Ballroom—Convention registration begins—Members and non-members—no registration fee

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Meetings of technical committees as then scheduled

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2 p.m.—Grand Ballroom—Convention called to order by President Robert F. Blanks

Concrete paving problems

Control of Scaling of Portland Cement Concrete Pavements Resulting from the Use of Chloride Salts—B. D. Tallamy, Supt. New York State Department of Public Works

SYNOPSIS—Traffic on heavily traveled roads requires faster ice removal than has been possible with salt-treated abrasives. As maintenance forces have struggled to meet the demand for uninterrupted service the use of straight salts has become increasingly common. Direct applications of as much as 600 lb of salt per two-lane mile have been used to combat extreme icing. Pavements constructed to withstand the weak brines deposited by salt-treated abrasives do not stand up under straight salt. New pavements appear more vulnerable than those two or more years old.

The observed resistance to salt action of the oil soaked center streak of uphill traffic lanes prompted New York state highway engineers to investigate the feasibility of the use of dilute oil applications as a protective measure. Laboratory tests indicate successful results may be expected. In the late summer of 1948 oil spray equipment was developed in time to treat nearly 60 miles of new concrete highways, which should provide a wide base for field observation of the effectiveness of the method.

*Formerly Hotel Pennsylvania

Experience with Air-Entraining Concrete in New Jersey—Charles M. Noble, State Highway Engineer

SYNOPSIS—Structurally sound portland cement concrete has not been a problem in New Jersey. Air-entraining concrete is used primarily to combat attack by de-icing chemicals. Damage to concrete New Jersey pavements is associated primarily with heavy axle loads. Except in isolated cases freedom from disintegration troubles is attributed to excellent aggregates, to rigid laboratory control and inspection of mix design, material, plant equipment and construction supervision on the job.

Air-entraining portland cement concrete was first specified for a concrete pavement contract in 1945 and since then has been used on many contracts with notable success to prevent deterioration due to scaling. It is not a "cure-all", and does not justify relaxing specifications, laboratory control or field inspection. The same high standards should be maintained as for ordinary concrete. Thus far New Jersey pavements of air-entraining concrete have not scaled or shown signs of disintegration from de-icing when chemicals have been properly mixed with abrasives at a rate not exceeding 75 pounds per cubic yard of abrasive. It is too early to form judgment but indications are the results achieved fully justify the use of air-entraining concrete.

What We Have Learned about Air-Entraining Concrete?—J. F. Barbee, Ohio Department of Highways

SYNOPSIS — A brief resumé of the hundreds of pages of published data on entrained air in concrete. Concrete with entrained air is more easily placed and has greater workability; in service it provides greater durability when exposed to severe weathering conditions. Air is entrained by using air-entraining cements or by adding air-entraining agents at the mixer. Amounts of air vary with a great number of factors but principally in proportion to the amount of agent used. Within limits durability to weathering increases with the amount of air entrained; compressive and flexural strengths are reduced as percentages of air increase. Entrained air is not a cure-all for failures of concrete durability but it does provide increased resistance to freezing and thawing and to the scaling action of de-icing chemicals. (Mr. Barbee's paper is scheduled for pre-convention publication in February).

Practices, Experiences and Tests with A-E Agents in Making Durable Concrete—R. F. Blanks and W. A. Cordon, U. S. Bureau of Reclamation.

SYNOPSIS—The authors discuss the advantages of air entrainment with regard to durability, permeability, workability, reduction in alkali expansion, time saving, reduction in water and cement, temperature rise, strength, abrasion resistance, monolithic lightweight concrete and mass concrete. Experiences are discussed with regard to pumping, transportation and placing, grading of aggregates, amount of air entrainment, factors affecting the amount of air entrained. Air-entraining agents and air-entraining cements are discussed briefly. Measurements of entrained air in concrete are considered briefly with a recommended procedure.

THURSDAY, FEBRUARY 24

9 a.m.—Grand Ballroom

Structural Problems

Rational Analysis and Design of Two-Way Concrete Slabs—C. P. Siess and N. M. Newmark, University of Illinois (The paper appears in the December JOURNAL.

SYNOPSIS—A new method for the design of two-way building slabs is proposed and its step by step development is described. A new moment distribution procedure is used to compute moments in a number of rectangular slabs continuous over rigid beams. Several variables are studied in the analyses and include: the ratio of sides, the effect of discontinuous edges, the torsional stiffness of the beam, various types of loading, and combinations of panels of various sizes and shapes. Certain conclusions are stated regarding the types of loading to be considered and the values of beam torsional stiffness to be assumed in the development of the design procedure.

Moments obtained in the foregoing analyses are modified to take into account the effect of additional variables. First,

the slab moments are increased by various amounts as a result of the deflection of the beams. Next, separate coefficients for dead and live load are replaced by combined coefficients for a live load-dead load ratio of 3.0. Then follows a study to determine the effect of discontinuous edges and this variable is eliminated from the procedure. Finally all slab moments are reduced by 20 percent in recognition of the redistribution of moments at high load. Additional studies are then made of the distribution of moments across the width of the slab and of the moments to be used for the design of the beams to complete the development of the design procedure.

A proposed design procedure is presented in specification form and is compared with other procedures of a similar nature and with the moment distribution procedure described in the appendix.

Reinforced Concrete Wall and Column Footings—Frank E. Richart, University of Illinois (The paper in 2 parts appears in the October and November JOURNALS.

SYNOPSIS—This paper reports an experimental investigation of reinforced concrete footings. The first extensive study of the subject since the work of Talbot in 1913; meantime many developments in materials, as well as in design methods.

Important from these tests are: (a) Definite proof that the tensile and bond resistance depend upon the bending moment and shear found by statics by consideration of the full applied load, and not 85 percent thereof, as assumed in current building codes; (b) Hooked ends of bars showed no particular advantage in bond resistance over straight bars, particularly when deformed bars of new improved types were used; (c) Hooked bars produced very little effect on resistance to diagonal tension, much less than is commonly assumed; (d) Welded mats proved particularly effective in resisting end slip of bars; (e) Footing caps or piers are effective in reducing the amount of reinforcement and in increasing the load capacity of footings; and (f) Diagonal tension seems to be the point of weakness in current design practice. The factor of safety of thin footings in this respect appears greater than in thick footings, and is generally greater in rectangular than in square footings when the conventional methods of computation are used.

The tests placed major emphasis on the isolated column footing. Principal attention has been given to the resistance of footings to failure by bond, diagonal tension and tension in the steel. Test footings were designed to produce these various types of failure. In addition, studies were made of the behavior of rectangular footings and footings having intermediate capitals or piers.

Experimental Aids in Structural Concrete Design—R. E. Glover, O. J. Olsen, and Carl Zangar, U. S. Bureau of Reclamation

SYNOPSIS — Statically determinate and indeterminate structures are generally designed by analytical methods, whether the structures are of steel, timber, or reinforced concrete. This paper describes several established experimental methods which are of considerable aid in the design and stress analysis of statically indeterminate structures at the laboratories of the Bureau of Reclamation. These methods are used when analytical procedures are too lengthy or involved or where there is no practical analytical approach to the problem. Described are photoelastic methods using both the polariscope and interferometer, the Beggs deformeter, Stresscoat and electric SR-4 strain gages. Applications are given for each method.

Noon—Luncheon—Georgian Room—President Blanks presents ACI awards

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2:30 p.m.—Grand Ballroom

Report of Annual Election

President's Address

Control in Ready Mixed Concrete Manufacture and Distribution — a 20-minute paper by Stanton Walker, Director of Engineering, National Ready Mixed Concrete Association; followed by discussion from users and manufacturers on problems in the production and use of ready mixed concrete.

Proposed Recommended Practice for the Application of Paint to Concrete Surfaces as reported by ACI Committee 616 and published in this JOURNAL (p. 353) will be presented in digest to the Convention on a motion for adoption as an ACI Standard.

FRIDAY, FEBRUARY 25

9 a. m. — Grand Ballroom — Research —an open session of Committee 115, Research—S. J. Chamberlin, Chairman; George W. Washa, Secretary. A group of short papers on research techniques and preliminary results. Papers and discussions not for publication or press release; no stenotype report.

2 p.m.—Grand Ballroom

Design Details for Architectural Concrete—J. J. Hogan, Portland Cement Association

SYNOPSIS—"To achieve beauty with concrete, as with any other material, requires studied design and skillful craftsmanship" are the words of architect Robert D. Murray. This paper directs attention to the matter of "studied design". The skilled craftsmanship required to construct architectural concrete must be preceded by the proper design executed by the architect and engineer. The designer must understand and appreciate the fundamental requirements of this type of construction. The paper discusses considerations appropriate to achieving the desired texture of surface as effected by form materials selected. Ornamentation should be considered in every detail from the standpoint of practicability in execution. Construction joints are inevitable and should be specified in logical locations. Similarly control joints should be specified with an understanding of where the most likely cracks due to volume change will occur and the design of reinforcement should be coordinated with the location of the control joints. Numerous illustrations support the text explanation.

Construction Practices for Architectural Concrete—E. B. Oberly, Portland Cement Association.

SYNOPSIS—Detailed planning and carefully drawn specifications executed by a contractor and superintendent with the know-how of form construction are necessary for good looking architectural concrete. This paper describes some of the more important construction practices to be considered to achieve the results desired. Form details and concrete quality are discussed in considerable detail, the former being supported by numerous illustrations.

Lightweight Aggregate Concrete—Ralph W. Kluge, Associate Professor structural engineering, Purdue University, Lafayette, Ind. (formerly of the National Bureau of Standards) M. M. Sparks, and E. C. Tuma, National Bureau of Standards.

Professor Kluge will present in digest the findings of a Bureau of Standards investigation for the Housing and Home Finance Agency of the federal government, to which a report has been made on the properties of selected lightweight aggregates and of concrete made from such materials. It includes information of wide general significance.

Progress in the Manufacture of Concrete Masonry Units—Jay C. Ehle, Cleveland Builders Supply Co. Mr. Ehle's paper is not in hand as this JOURNAL goes to press.

☆ ☆ ☆

NEW YORK IN FEBRUARY

Make your personal plans now to attend the 45th Annual Convention in New York City. The big-town meetings will be held at Hotel Statler* . . . make room reservations direct to hotel.

Daytime meetings only—February 23, 24, 25

*Formerly Hotel Pennsylvania

☆ ☆ ☆

Who's Who This Month

Committee 616—Recommended Practice for the Application of Paint to Concrete Surfaces

Committee 616 has completed its report "Proposed Recommended Practice for the Application of Portland Cement Paint to Concrete Surfaces" which is published on p. 353 as released by the Standards Committee for consideration for adoption as an ACI standard at the forthcoming convention in New York in February. This report is a revision of a previous report of Committee 616 published for information and discussion in June 1942 which had a wide popular demand requiring several reprintings.

The recently reorganized Committee is headed by Chairman G. E. Burnett, Bureau of Reclamation, Denver. Able co-members are F. O. Anderegg, John B. Pierce Foundation; T. E. Chisholm, Hydro-Electric Power Comm. of Ontario; R. E. Copeland, National Concrete Masonry Assn.; W. E. Groben, U. S. Forest Service; Bernard Malm, Sears, Roebuck & Co.; G. S. Paxson, Oregon State Highway Dept., and Clara Sentel, National Bureau of Standards.

Boyd S. Brooks

Boyd S. Brooks is a graduate of the Johns Hopkins University, class of 1929, with a degree of Bachelor of Engineering (in civil engineering). After a year with the state highway commission of Maryland on road and bridge construction, he went with the Division of Tests, Public Roads Administration. In 1939 he went on active duty as a Lieut. (j.g.), Civil Engineer Corps, USNR, to assist in equipping and outfitting the Navy's new model testing basin at Carderock, Maryland. After the Taylor Model Basin was placed in operation in 1940, he continued as Public Works Officer until 1944, when he was detached for duty with the Navy's construction battalions. Comdr. Brooks was the Officer-in-Charge of the 56th Naval Construction Battalion on Guam and later he inactivated the 85th NCB on Wake Island. On his return to the states he transferred from the Reserve to USN and for the past two years he has been Executive Officer of the U. S. Naval School, CEC Officers, at Port Hueneme, California. He has been a member of the Institute since 1946.

Michel Bakhoum

Michel Bakhoum appears as a JOURNAL author for the third time within a year with the publication this month of "Direct Dimensioning of Regular Sections", p. 381. Previous papers on design appeared in February 1948 and April 1948. Mr. Bakhoum is returning this month to his home in Egypt where he is Lecturer at Fouad I University, Giza. He has been at the University of Illinois since February 1946 as "mission member sent by the Egyptian government".

S. O. Asplund

S. O. Asplund, a newcomer to the American Concrete Institute was graduated from the Royal Institute of Technology at Stockholm, Sweden in 1924 with a degree of C.E.; in 1943 he received his D. Eng. at the same university on his treatise "On the Deflection Theory of Suspension Bridges". Dr. Asplund's experience included employment with the American Bridge Company, the North Carolina Highway Department, and the Cleveland Terminal Association from 1925 to 1929. As a contractor in Sweden since 1931 he has designed and built several flexible-type suspension bridges with concrete decks as well as numerous concrete structures and conventional concrete and steel bridges. He is a lecturer on bridge design at the Royal Insitute of

Technology, Stockholm, and a member of the Swedish Academy of Engineering Sciences. In addition to ACI, he is a member of the Swedish Concrete Association, the International Association for Bridge and Structural Engineering and the A.S.C.E. He is the author of about 30 engineering papers; "Strengthening Bridge Slabs with Grouted Reinforcement", p. 397, is his first contribution to American technical literature.

Milton Brumer

Milton Brumer expands on the consistency of design by the plastic theory and resultant economies of its use in his paper "Comparative Designs of a Segmental Skewed Frame Concrete Bridge by the Straight Line and Plastic Theory Methods". As principal engineer with O. H. Ammann, Consulting Engineer, New York, he was in charge of the design of about seventy-five concrete rigid frame parkway and highway bridges. Formerly he assisted in or directed the preparation of plans for numerous bridge and vehicular tunnel projects for The Port of New York Authority, The Pennsylvania Turnpike Commission, Ralph Smillie, Consulting Engineer, New York City, Parsons, Klapp, Brinckerhoff and Douglas, and Knappen Engineering Company.

Mr. Brumer was graduated from the Rensselaer Polytechnic Institute in 1923 and received the degree of Master of Science in Mining Engineering from the University of Alabama in 1924. He is now associate partner of the firm Ammann and Whitney, Consulting Engineers. He has been an ACI member since 1946, and is a member of Committee 314 on Concrete Frame Bridges, a member of the A.S.C.E., Sigma Xi and a registered professional engineer in the States of New Jersey, New York and Pennsylvania.

The portland cement industry in continental U. S. shipped 185,491,000 barrels of cement in 1947.

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New Members

Massachusetts was in the lead with seven new member applicants in November; New York and Indiana were tied with four each. The 46 member applicants in November approved by the Board of Direction are listed—bringing ACI's membership total to 4364, December 1.

Arkansas

Burgess, Woodrow L., Mountain Home, Ark.

California

Rodriguez, Pedro, International House, Berkeley 4, Calif.

Wygant, R. D., c/o O Inc-NOy Contracts, FPO 926, c/o P. M., San Francisco, Calif.

Colorado

Burrows, Richard W., U. S. Bureau of Reclamation, Bldg. 1B, Denver Federal Center, Denver, Colo.

Wolf, William H., 6008 E. 29th Ave., Denver 7, Colo.

Connecticut

Mirsky, Aron L., 417 Garden St., Hartford 5, Conn.

Indiana

Lewis, Donald W., Joint Highway Research Project, Purdue University, Lafayette, Ind.

Scheid, Dana A., 747 S. Courtland Ave., Kokomo, Ind.

Stanley, Adrian L., R. R. 3, Box 128A, Terre Haute, Ind.

Teran, Temistocles, P. O. Box 104, University of Notre Dame, Notre Dame, Ind.

Massachusetts

Crosier, Clayton M., 209 Fuller St., Dorchester 24, Mass.

Dwyer, John J., 109 Fuller St., Dorchester 24, Mass.

Guardia, Gabriel J. de la, The Graduate House, Cambridge 39, Mass.

Lawrence, Eugene F., 80 Palmer Ave., Falmouth, Mass.

Magee, Robert L., c/o Haller Testing Laboratories, Inc., 104 Cummington St., Boston, Mass.

Venegas, Emilio Jose, Box 538, M. I. T. Dorms, Cambridge 39, Mass.

Whitney, Philip C., 325 Commonwealth Ave., Boston, Mass.

Michigan

Ellsberg, Harry, 1256 National Bank Bldg., Detroit, Mich.

Mississippi

Crawley, Walter O., Box 204, Clinton, Miss.

Missouri

Kotakis, Dimitro A., 512 S. 5th, Columbia, Mo.

New York

Fisher, Joel E., Jr., 1020 Fifth Ave., New York 28, N. Y.

Toepp, Vincent A., 707 W. Liberty St., Rome, N. Y.

Ohio

American Fluresit Co., 4011 Red Bank Rd., Cincinnati, Ohio (W. M. Spurgeon)

Roberts, D. G., Floyd G. Browne & Assoc., 196 S. Main St., Marion, Ohio

Pennsylvania

Hankin, Bernard, Marchwood Apts., 5515 Wissahickon Ave., Philadelphia 44, Pa.

Lessig, Donald P., 1507 Summit St., McKeesport, Pa.

Schulze, Axel, 10 S. 18th St., Philadelphia 3, Pa.

Utah

Moser, Erwin U., 426 N. Main, Salt Lake City 3, Utah

Virginia

Roberts, John W., P. O. Box 1J, Richmond 1, Va.

Wisconsin

Clore, Robert H., 306 Bridge St., Cedarburg, Wis.

Canal Zone

Municipal Engineering Division, The Panama Canal, Balboa Heights, C. Z. (F. H. Lerchen)

Hawaii

Tanaka, Manabu, P. O. Box 282, Paia, Maui, T. H.

Puerto Rico

Rosso, Jose E., No. 556 Miramar Ave., Apt. 5, Santurce, Puerto Rico

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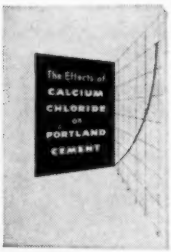

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- The effect on volume change?... surface wear?

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Company.....

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City.....State..... JC-1

Canada

Pybus, Ralph C., 700 Taylor St., Vancouver, B. C., Canada

Cuba

Hernandez, Alfredo Perez, Villa Alicia, Madruga, Havana, Cuba

Jimenez, Jose Miguel Gonzalez, Villegas 466 Apt 401, La Habana, Cuba

Marina, David Jorge, 5 No. 364 entre 16 y 18 R., Almendares, Marianao, La Habana, Cuba

Mexico

Pena, Antonio Ruiz de la, Lopez Cotilla 383, Guadalajara, Jal., Mexico

England

Krishnamacharyulu, K., 20, Addiscombe Grove, East Croydon, Surry, U. K., England

The Reinforced Concrete Association, 94/98 Petty France, London S. W. 1, England (R. V. Chate)

New Zealand

Cooper, E. R., Dominion Physical Laboratory, Lower Hutt, N. Z.

Norway

Hoff, Ingolf, N. T. H., Trondheim, Norway

Netherlands West Indies

Strode, Willard, Box 652, Lago Colony, Aruba, Curacao, N. W. I.

Spain

Semelas, Alfredo, Villanueva 17, Madrid, Spain

Sweden

Asplund, Sven Olof, Orebro, Sweden

Yugoslavia

Juranovic, Vladimir, Kaciceva 26, Zagreb I, Yugoslavia

Hotel Pennsylvania became Hotel Statler January 1

The decision to change the name of the Hotel Pennsylvania to Hotel Statler after it had been purchased from the Pennsylvania Railroad came about after the management asked the opinion of about 10,000 guests. The guests voted 3 to 1 for Hotel Statler. All that is involved is a change in name. Statler has always operated Hotel Pennsylvania.

Positions and Projects — ACI Members

Harrison F. Gonnerman promoted—Hubert Woods goes to PCA

ACI past president, Harrison F. Gonnerman, Director of Research, Portland Cement Association, on December 1 was advanced to the position of Assistant to the Vice President for Research and Development, succeeding another ACI past president, Franklin R. McMillan, who had held that post since January 1947. (See page 10, November News Letter announcing Mr. McMillan's retirement).

Long active in ACI affairs (since 1918) Mr. Gonnerman is well known as an author of JOURNAL papers. He has twice received the Wason Medal for noteworthy research. In addition to ACI he is a member of A.S.T.M., the Western Society of Engineers, A.S.C.E., Tau Beta Pi and Sigma Xi. He holds a B.S. degree in C.E. and an M.S. from the University of Illinois.

Hubert Woods also a long time ACI member (20 years) succeeds Mr. Gonnerman as Director of Research. He has been Research Director of the Riverside Cement Co. of Los Angeles since 1926 and will assume his new duties early in January 1949. Mr. Woods is the author of various technical papers on cement and cement manufacture. He is a member of Sigma Xi and A.S.T.M. and a member of the P.C.A. Advisory Committee for the Long Time Study of Cement Performance in Concrete. He is a graduate from the California Institute of Technology in 1923 with a B.S. in Chemical Engineering.

T. D. Mylrea

T. D. Mylrea, head of the department of civil engineering in the University of Delaware School of Engineering since 1934, has relinquished the chairmanship to Dr. Julian W. Shields, professor of theoretical and applied mechanics, because of ill health. Professor Mylrea, retains his instructional work and his rank as professor.

Long active in ACI, Professor Mylrea is the author of a number of Institute papers and is a member of Committees 115, 315 and 318.

J. C. Witt

J. C. Witt, long time ACI member (1924) and for the past ten years technical director of Marquette Cement Manufacturing Co., has entered private practice as a consulting engineer. Mr. Witt received his Ph.D. degree in chemistry and physics at the University of Pittsburgh, and later was graduated in mechanical engineering at Armour Institute of Technology. He received his professional M.E. degree from Armour.

He started his career in the cement industry as technical director of the Rizal Cement Co., in the Philippine Islands. Prior to joining the Marquette organization in 1938, Mr. Witt was director of research for Universal Portland Cement Co. and its successor, Universal Atlas Cement Co. His activities have included prospecting for raw materials, plant operation, experimental engineering and research. He is ACI representative on subcommittee V of A.S.T.M. Committee C-9.

Harry C. Shields

ACI Member Harry C. Shields has been appointed Eastern manager of the construction division of The Presstite Engineering Co., St. Louis. Formerly manager of the technical bureau of Pennsylvania-Dixie Cement Corp., New York, N. Y., and field engineer and sales manager of the Marquette Cement Manufacturing Co., Chicago, Mr. Shields has had 26 years' experience in the cement and construction industries.

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Stephen W. Benedict

Stephen W. Benedict, formerly materials engineer in the cement and concreting materials section of the National Bureau of Standards, has been appointed director of research of the Master Builders Co., Cleveland. Before joining the Bureau of Standards staff Mr. Benedict was a materials engineer with the Illinois division of highways and later a structural engineer with the National Advisory Committee for Aeronautics. He is a graduate in civil engineering from the University of Illinois with post graduate work in law and mathematical statistics. He has been an ACI member since 1941 and is also a member of A.S.T.M.

Waldo G. Bowman

Waldo G. Bowman, ACI member since 1940, editor of *Engineering News-Record* and *Construction Methods*, has been nominated for election as Director, District 1 of the A.S.C.E.

Hubert Rusch

Hubert Rüsch, a member of the ACI, recently was appointed Head of the Department for Bridge Construction and Reinforced Concrete at the Institute of Technology of Munich, Germany. In this new position, Dr. Rüsch will be head of the Government testing laboratory connected with the Institute. He is an Austrian by birth and was associated with design and construction of outstanding reinforced concrete structures in Europe, England and South America. In recent years he lead in the development of shell structures, prestressed concrete work and precast construction projects of Dyckerhoff and Widmann. Just prior to World War II the Franklin Institute of the State of Pennsylvania awarded him the Longstreth medal in recognition of his work.

Portland Cement Association elects new board chairman

The president of the Peerless Cement Corp., Walter C. Russell, was elected chairman of the Board of Directors of the Portland Cement Association at a recent annual meeting in Chicago, succeeding Joseph S. Young of Allentown, Pa., president of the Lehigh Portland Cement Co. Peerless holds two contributing and one corporation membership in ACI and Lehigh three contributing memberships dating since 1924 and one corporation membership since 1905.

45th
Annual Convention
American Concrete Institute
NEW YORK CITY
Feb. 23-25, 1949

Honor Roll

February 1 to November 30, 1948

The Morgan father and son contest between Illinois and Wyoming shows up in the two top places on the Honor Roll to the end of November. The next notable contest is in California where Raymond E. Davis and Thomas E. Stanton are in third and fourth places respectively. Howard Simpson in Massachusetts is in fifth place.

The yearly record will close with results received by ACI up to January 31, 1949.

Newlin D. Morgan (Ill.).....33
Newlin D. Morgan, Jr. (Wyo.).....30
Raymond E. Davis (Calif.).....15
Thomas E. Stanton (Calif.).....14
Howard Simpson (Mass.).....10
James A. McCarthy (Ind.)..... 7½
Leon Venegas (Costa Rica).....7
Henry L. Kennedy (Mass.).....6½
Raphael Callejas H. (Guatemala).....6
Harrison F. Gonnerman (Ill.).....6
Oliver G. Julian (Mass.)..... 5
Wm. H. Thoman (Colo.).....4½
Harold S. Carter (Utah).....4
E. I. Fiesenheiser (Ill.).....4
Thomas C. Kavanagh (Pa.).....4
R. H. Sherlock (Mich.).....4
Lewis H. Tuthill (Colo.).....4
Jose Antonio Vila (Cuba).....4
Phil M. Ferguson (Texas).....3½
Elmo C. Higginson (Colo.).....3½
Myle J. Holley, Jr. (Mass.).....3½
A. E. Mackney (Canada).....3½
J. Antonio Thomen (Dominican Republic).....3½
Bailey Tremper (Wash.).....3½
G. E. Troxell (Calif.).....3½
Stanton Walker (D. C.).....3½
Robert W. Freeman (Calif.).....3
Ray C. Giddings (Calif.).....3
John C. King (Ohio).....3
George E. Large (Ohio).....3
F. N. Menefee (Mich.).....3
Robert B. B. Moorman (Mo.).....3
Rene S. Pulido y Morales (Cuba).....3
Robert Morris (Colo.).....3
James M. Polatty (Ga.).....3
Richard A. Roberts (Calif.).....3
Joseph J. Waddell (Calif.).....3
Alberto Dovali Jaime (Mexico).....2½
R. R. Kaufman (Ohio).....2½
George L. Otterson (Va.).....2½
Frank E. Richart (Ill.).....2½
John A. Ruhling (Ill.).....2½
John C. Sprague (Ga.).....2½
H. D. Sullivan (Wash.).....2½
Boyd G. Anderson (N. Y.).....2
Hugh Barnes (Calif.).....2
F. Campus (Belgium).....2
Jose Luis Copacete (Puerto Rico).....2
John K. Class (Pa.).....2
Moses D. Cohan (N. Y.).....2
W. S. Cottingham (Wis.).....2
Jacob J. Creskoff (Pa.).....2
H. J. Gilkey (Iowa).....2
Ernst Gruenwald (N. Y.).....2
Edward L. Howard, Jr. (Calif.).....2
William E. Lumb (Canada).....2
F. T. Mavis (Pa.).....2
Eugene Mirabelle (Mass.).....2
Hugh E. Odor (Wash.).....2
Walter H. Price (Colo.).....2
Manuel Ray Rivero (Cuba).....2
John W. Robison (Colo.).....2
Georges Ronai (Turkey).....2
Thomas C. Shedd (Ill.).....2
C. H. Scholer (Kans.).....2
Edward W. Scripture, Jr. (Ohio).....2
E. Copeland Snelgrove (England).....2
J. Neil Thompson (Texas).....2
E. W. Thorson (Colo.).....2
Juan Agustin Valle (Argentina).....2
C. D. Williams (Fla.).....2
Harold W. Brewer (Colo.) 1½
R. V. Chate (England).....1½
A. R. Collins (England).....1½
R. Torres Colondres (Puerto Rico).....1½
Harmer E. Davis (Calif.).....1½
George C. Ernst (Nebr.).....1½
Milton Fromer (Calif.).....1½
Francis R. McCabe (Ore.).....1½
W. E. Parker (Canada).....1½
Dean Peabody, Jr. (Mass.).....1½
Jerome M. Raphael (Colo.).....1½
Clarence Rawhouser (Colo.).....1½
Anton Rydland (Calif.).....1½
Roy T. Sessums (La.).....1½
James M. Shivley (Ark.).....1½
Harry F. Thomson (Mo.).....1½
John Tucker, Jr. (D. C.).....1½
Calvin T. Watts (La.).....1½
Harold E. Wessman (Wash.).....1½
Ray A. Young (Wash.).....1½
Jerome O. Ackerman.....1
J. B. Alexander.....1
W. H. Allison.....1
A. Amirikian.....1
Edwin C. Anderson.....1
J. A. Bakker.....1
George E. Barnes.....1
Paul H. Barnes.....1
S. B. Barnes.....1

F. Barona....1
Frank H. Beinhauer....1
Rene L. Bertin....1
Robert F. Blanks....1
A. J. Boase....1
E. K. Borchard....1
H. L. Bowman....1
H. E. Burr....1
John Butkus....1
A. Carbone....1
A. N. Dutta Choudhury....1
C. O. Christy....1
Miles N. Clair....1
Rodger B. Collons....1
R. E. Copeland....1
Jack E. Counts....1
Walter Dardel....1
J. Slater Davidson, Jr....1
A. Finlayson....1
T. H. Fleming....1
R. E. Fowle....1
Zorislav Franjetich....1
Bengt Friberg....1
Frank M. Fucik....1
Carmelo Galindez....1
Anthony J. Giardina....1
Hernando Gonzalez....1
R. R. Gore....1
Emil A. Gramstorff....1
Eugene Guillard....1
Elliot A. Haller....1
George E. Hatch....1
F. E. Henman....1
Samuel Hobbs....1
W. C. Huntington....1
Phil C. Huntly....1
Wm. R. Johnson....1
H. D. Jolley....1
Edward F. Keniston....1
P. J. Kennedy, Jr....1
M. B. Lagaard....1
Orla A. Larsen....1
B. Leon....1
W. G. Limbach....1
L. D. Long....1
Inge Lyse....1
Charles Macklin....1
Donald R. MacPherson....1
John V. Maescher....1
Clyde P. Mason....1
Sam McCluer....1
William G. McFarland....1
Douglas McHenry....1
F. R. McMillan....1
Carl A. Menzel....1
L. J. Meszaros....1
Oliver H. Millikan....1
Leonard J. Mitchell....1
John J. Mullen....1
R. P. Mumford....1
T. D. Mylrea....1
Amado B. Navoa....1
William T. Neelands....1
C H. Neill....1
N. M. Newmark....1
D. G. Norman....1
Wm. D. Nowlin....1
Calvin C. Oleson....1
Douglas E. Parsons....1
Charles W. Payne, Jr....1
R. A. Plumb....1
James J. Pollard....1
Horace A. Pratt....1
Abdul Qayum....1
R. D. Rader....1
Thomas J. Reading....1
J. W. Rivett....1
H. H. Roberts....1
Fernando A. Rojas B....1
E. Rosecrans....1
H. C. Ross....1
E. Gonzalez Rubio....1
C. E. Sandstedt....1
Edgar A. Sealey....1
C. Seshavatharam....1
Joseph Sill....1
Francis P. Sloan....1
Gustave Spirz....1
Hale Sutherland....1
E. O. Sweetser....1
Howard M. Thomas....1
T. Thorvaldson....1
Dean P. Tsagaris....1
Oscar J. Vago....1
Walter C. Voss....1
Jukka E. Vuroinen....1
W. F. Way....1
Paul Weidlinger....1
J. M. Wells....1
H. Roger Williams....1
Piers M. Williams....1
Merle D Wilson....1
Francis P. Witmer....1

The following credits are, in each instance "50-50" with another member.

Robert W. Abbett
George C. Alden
A. A. Anderson
Oscar Vega Arguelles
Manuel A. Arosemena
S. Balasubranmanyam
A. Allan Bates
E. W. Bauman
C. T. Bishop
Petrus Manzano Bonilla
P. G. Bowie
A. R. Brickler
Boyd S. Brooks
L. Rees Brooks
Sterling Lowe Bugg
R. A. Burmeister
B. W. Burns
Jose Luis Casas
Thomas F. Chace
Wilbur H. Chamberlin
Richard de Charms
J. E. Christiansen
James H. H. Chun
H. W. Chutter
Emil W. Colli
F. Coppen
Rolland Cravens
J. M. Crom
R. A. Crysler
Rembert L. Curry
Philip Davis
Charles W. De Groff
John G. Dempsey
R. F. Dierking
J. B. Dolsen
J. E. Dunn
J. Dvorak
John N. Eckle
Harlan H. Edwards
A. C. Eichenlaub
F. L. Ehasz
Arthur D. Engle
William E. Feidt
Paul Fischer
Rudolph Fischl
E. M. Fleming
Frederick E. W. Fowler
Jaichand Hirachand Gandhi
Leo V. Garrity
Carl V. Gezelius
O. H. Gosswein
Howard A. Gray
Luis Alberto Truque Gurdian
J. R. G. Hanlon
Bernard Harrison
James F. Hartman
Frank W. Haws
H. L. Henson
A. E. Hiscox
W. C. Hodgkins
John Hogan

G. W. Holcomb
H. N. Howe
Walter L. Huber
J. W. Hubler
Manuel Castro Huerta
Robert B. Hyslop
C. L. N. Iyengar
Orville E. Jack
Frank H. Jackson
M. E. James
Bruce Jameyson
E. B. Johansen
Samuel Judd
Harry D. Jumper
J. W. Kelly
Thomas M. Kelly
John J. Kennedy Jr.
K. J. Krawczyk
H. M. Larmour
Robert E. Lee
L. G. Lenhardt
John F. Long
Wallace A. Marsh
Hubert F. McDonell
Juan B. McKay
Callix E. Miller, Jr.
R. E. Mills
Hugh Montgomery
Ray M. Moorhead
E. J. Mueller
A. E. Myers
Israel Narrow
Henry L. Neve
Elmer H. Oechsle
Jaime Olaechea
Francisco Santos Oliva
Chester B. Palmer
G. S. Paxson
Donald O. Peck
Gerald Pickett
Ben Poisner
Chesley J. Posey
Emil H. Praeger
Herman G. Protze
Warren Raeder
R. C. Rao
Raymond C. Reese
Dwight F. Roberts
Bernard L. Robinson
Fredric Rusche
Fred H. Ryan
Fernando Sanchez
Edward B. Schicker, Jr.
Herman Schorer
Joseph J. Shideler
Charles A. Shirk
Chester P. Siess
R. Simons
J. Singleton-Green
Charles M. Spofford
Edwin A. T. Spuehler
Byram W. Steele
A. R. Stuckey
M. O. Sylliassen
Ruth D. Terzaghi
A. G. Timms
Myron H. Walters
Donald R. Warren
J. C. Watt
J. D. Whittaker
William H. Wisniski
Harry C. Witter
K. B. Woods
Roderick B. Young

Marquette Cement Manufacturing Co. awarded gold "Oscar"

Marquette Cement Manufacturing Co., Chicago, was recently awarded the gold "Oscar", top award for excellence in annual reporting, for its annual report for 1947. This report was chosen from more than 4000 corporate reports by an independent board of judges in the national survey conducted annually by Financial World magazine. W. A. Wecker, president of Marquette received the award, on behalf of his company, from the hands of Donald V. Fraser, president of Missouri-Kansas-Texas Railroad, last year's winner.

India proposes the establishment of a central road research institute

The Council of Scientific and Industrial Research, New Delhi, India, has published a tentative scheme for the establishment of a Central Road Research Institute, for which there is a great need in developing a highly efficient system of roads important to the prosperity and greatness of the nation. The proposed laboratory and experiment stations will undertake research work to improve existing techniques of construction and maintenance, to investigate soils with a view to evolve a cheap type of rural road, to investigate problems of road safety and to develop improved types of road vehicles which would be efficient but less destructive of rural road surfaces.

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Construction Methods, Supervision

Frederick Wilhelms, Sr.

Word recently has been received of the death of Frederick Wilhelms, Sr., president and treasurer of the Wilhelms Construction Co., Elizabeth, New Jersey since 1923. Mr. Wilhelms was also President of the F. C. G. Co., a contractor's equipment rental organization and a member of the Board of Trustees of the Building Contractors Association of New Jersey. He had been a member of the Institute since 1941.

Ralph D. Rader

Ralph D. Rader, a member of the Institute since 1944 and manager of the Portland Cement Information Bureau in San Francisco died at his home in Berkeley, November 12. He was a native of Madison, Ohio, and a graduate of the University of Florida. He served as captain, Corps of Engineers in France in World War I and became associated with the Montana State engineering staff in 1920. He was State Engineer of Montana from 1927 to 1936 when he became associated with the Portland Cement Information Bureau. He was active in the Structural Engineers Association of Northern California, the A.S.C.E. and had served as secretary of the Concrete Masonry Association of Northern California.

New pavement committee meets

Committee 325, Structural Design of Concrete Pavements for Highways and Airports, under the chairmanship of L. W. Teller, made a propitious start on its assignment at a meeting in Washington, D. C. at the National Academy of Sciences Building December 3.

An idea of the important and ambitious program of the committee is indicated by its assignment:

"The committee's basic assignment is to develop a recommended practice for the design of structural details for concrete pavement for highway and airport service—slab dimensions, cross-sections, joints, reinforcement, etc., on the basis of the best information available, and to keep such recommendations up to date as new information indicates need for revision."

Henry Aaron, Civil Aeronautics Administration, was made secretary of the committee and a procedural program involving the appointment of appropriate subcommittees was outlined.

The committee itself will undertake no new research. Each subcommittee will be charged with keeping abreast of research being done by other agencies, supplying information in relation to the assignment of particular subcommittees.

Some of the topics to be considered are indicated in the following outline:

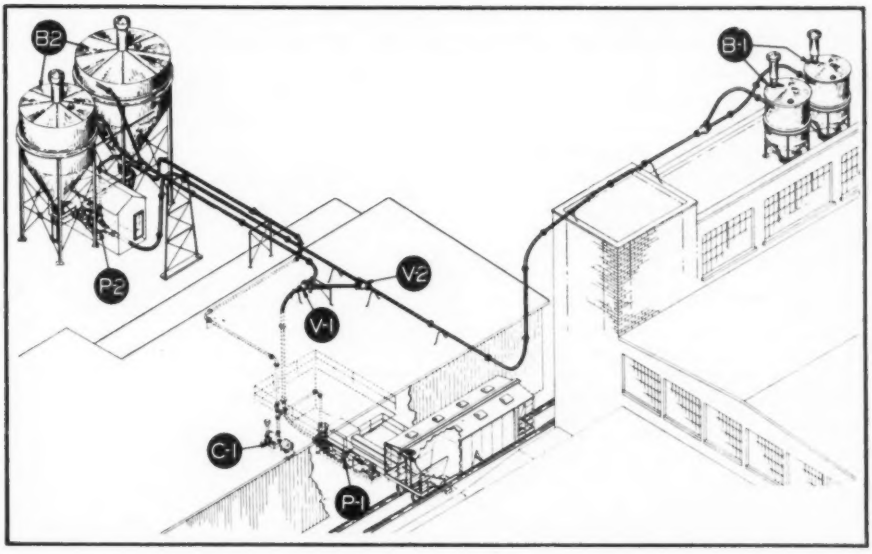
Structural Design of Concrete Pavements for Highways and Airports

1. Bases and subgrades
   a. Effect of subgrade quality on pavement slab design.
   b. Stabilized bases for concrete pavements.
   c. Methods for measuring supporting value of subgrades and bases.
2. Pavement slab dimensions.
   a. Fundamental considerations.
   b. Length.
   c. Width.
   d. Thickness and cross-section design.
3. Structural design of joints.
   a. Functions of joints (expansion, contraction, warping).
   b. Load transfer—its various functions.
   c. Design of load transfer systems in several types of joints.
   d. Fillers and seals.
4. Steel reinforcement.
   a. Function.
   b. Effect on slab design.
   c. Types.
   d. Design.

The chairman and ten members of the committee were present at the initial meeting.

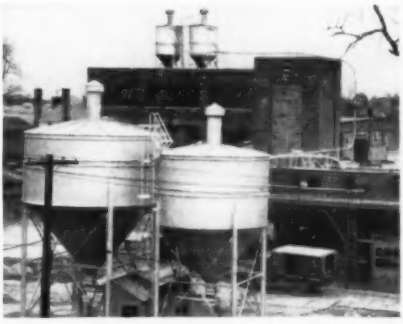
NOTICE!

Copies of original books and magazine articles reviewed in the Current Reviews section of the JOURNAL are **not** available from ACI. The footnote formerly published with that section indicated that separate prints of *the reviews section itself* are available, not separate prints of the individual articles reviewed, as some readers have assumed.



F-K SYSTEMS

unload from cars, convey to process or storage... reclaim from storage and convey to process



The Fuller-Kinyon installation illustrated is used for unloading and conveying Portland cement and filler dust in an asbestos shingle plant. These systems, installed in an existing plant, were designed and built to service process equipment and provide for future expansion.

One system (P-1) unloads materials from hopper bottom cars and conveys, through the use of two-way diverting valves (V-1 and V-2), direct to process bins (B-1) on top of building, or to reserve storage bins (B-2) in the yard. A second system (P-2) conveys materials from the storage bins (B-2) to process bins (B-1). A very flexible and simple system, economically possible only with Fuller-Kinyon.

A Fuller Rotary Single-stage Compressor (C-1) furnishes air for either conveying system, capacity 282 c.f.m., 25-lb. pressure. Air, where and when needed, and at the right pressure to do the work most economically and efficiently.

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SYNOPSES of recent ACI Papers and Reports

Institute papers of this JOURNAL Volume 20 are currently available at prices indicated. Please order by title and title number.

RECOMMENDED PRACTICE FOR WINTER CONCRETING METHODS (ACI 604-48).....45-1

Price 50 cents.

REPORT of COMMITTEE 604—Sept. 1948, pp. 1-20, (V. 45) in special covers

Supersedes 44-13.

This ACI standard establishes methods of cold-weather concreting for thin sections and mass concrete. Heating of materials, accelerators and anti-freezes, curing and temperature records during curing, subgrade (or base) preparation, protective coverings during curing, and form removal are discussed for both types of job, and preferred methods are indicated. An appendix entry outlines objectives of the special winter methods with background material which indicates the "why" of some of the recommended practices. Charts in the appendix indicate effect of curing temperature on concrete strength, and a list of 135 selected references to periodical literature on winter concreting methods is included.

RELATIVE STRENGTHS OF PORTLAND CEMENT MORTAR IN BENDING UNDER VARIOUS LOADING CONDITIONS.....45-2

Price 35 cents.

JACOB PORTER FRANKEL—Sept. 1948, pp. 21-32 (V. 45)

Assuming the behavior of standard mortar to be similar to that of plain concrete, tests were performed on 99 small mortar beams under sixth-, third- and center-point loading to verify the applicability of the statistical theory of the strength of brittle materials to concrete structures. The theory, as first developed by Weibull, is briefly analyzed, compared to the methods used by Tucker, and finally applied to the tests at hand. Agreement between the theoretical and experimental findings is so close as to warrant future testing on plain concrete specimens.

TERRAZZO AS AFFECTED BY CLEANING MATERIALS.....45-3

Price 35 cents.

D. W. KESSLER—Sept. 1948, pp. 33-40 (V. 45)

Effect of cleaning materials on terrazzo was studied using solutions of the following detergents: soda ash, trisodium phosphate and synthetic sulfonate. Seventy-six different aggregates were used in preparing terrazzo disks which were moist cured 3 months and then surface ground to form a shallow dish. Detergent was allowed to stand in dish 30 minutes before rinsing and drying (at 105 C). This testing cycle was repeated until deterioration tendencies were established. Soda ash solution proved much more destructive than the trisodium phosphate; however, up to the time of writing (after 850 cycles) the synthetic sulfonate had caused no failure.

TRANSPORTING READY-MIXED CONCRETE IN OPEN DUMP TRUCKS.....45-4

Price 35 cents.

R. A. BURMEISTER—Sept. 1948, pp. 41-56 (V. 45).

Milwaukee experience in hauling air-entraining concrete in open dump truck bodies is recorded. Slump, air content, specific weight, compressive strength and workability tests were made on concrete before and after a trial run over bumpy pavement. Some similar tests were made on concrete being placed in a Milwaukee street after open truck transportation. The author concludes that open trucks for hauling ready-mixed air-entraining concrete are satisfactory when air content is between 3 and 6 percent (5 to 6 sacks cement per cu yd) and hauling time is 45 minutes or less, provided that aggregate moisture, grading and slump are carefully controlled.

A STUDY OF ALKALI-AGGREGATE REACTIVITY BY MEANS OF MORTAR BAR EXPANSIONS.....45-5

Price 35 cents.

T. M. KELLY, L. SCHUMAN and F. B. HORNIBROOK—Sept. 1948, pp. 57—80 (V. 45)

Many types of aggregate were combined in varying amounts and sizes with high- and low-alkali cements and formed into 1 x 1 x 10-in. mortar bars. The bars were stored either at 70 F or at 100 F and their expansions measured at ages ranging from 1 month to 4 years.

In combination with high-alkali cements, opal, opaline chert and a siliceous dolomitic limestone were found to cause greatest expansion. Certain aggregates containing volcanic glasses and some natural sands and gravel also caused excessive expansion; with one exception, these sands contained small amounts of opal.

Greatly delayed expansion resulted with the very fine sizes of opal, particularly in combination with high-soda cement. Similar behavior resulted with minus No. 81 size opal and low-alkali cement with either Na_2SO_4 or K_2SO_4 additions.

Materials such as dehydrated kaolin, soda feldspar, magnesium fluosilicate, acetic acid and calcium hydroxide added in small amounts as correctives were ineffective. However, diatomaceous earth in sufficient quantity as a cement replacement eliminated expansion.

REINFORCED CONCRETE WALL AND COLUMN FOOTINGS PART I.....45-6a

Price 60 cents.

FRANK E. RICHART—Oct. 1948, pp. 97-128 (V. 45)

PART II.....45-6b

Price 35 cents.

FRANK E. RICHART—Nov. 1948, pp. 237-260 (V. 45)

This paper presents, in two parts, a report on an experimental investigation of reinforced concrete footings. It represents the first extensive study of the subject since the well-known work of Talbot in 1913. In the meantime there have been many developments in materials, as well as in design methods.

Important developments from these tests include the following: (a) Definite proof that the tensile and bond resistance depend upon the bending moment and shear found by statics by consideration of the full applied load, and not 85 percent thereof, as assumed in current building codes; (b) Hooked ends of bars showed no particular advantage in bond resistance over straight bars, particularly when deformed bars of new improved types were used; (c) Hooked bars produced very little effect on resistance to diagonal tension, much less than is commonly assumed; (d) Welded mats proved particularly effective in resisting end slip of bars; (e) Footing caps or piers are effective in reducing the amount of reinforcement and in increasing the load capacity of footings; and (f) Diagonal tension seems to be the point of weakness in current design practice. The factor of safety of thin footings in this respect appears greater than in thick footings, and is generally greater in rectangular than in square footings when the conventional methods of computation are used.

In these tests, major emphasis has been placed on the isolated column footing. Principal attention has been

Continued on p. 20

ACI publications in large current demand

Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-48)

Approved as a Standard of the American Concrete Institute at its 44th Annual Convention February 25, 1948. Ratified by letter ballot June 29, 1948. It is of large format, bound to lie flat and presents typical engineering and placing drawings with discussion calling attention to important considerations in designing practice. It was prepared to simplify, speed, and effect standardization in detailing. It is believed to be the only publication of its kind in English. It is meeting wide acclaim among designers, draftsmen and in engineering schools. Price—$3.00, to ACI Members—$1.75.

ACI Standards—1948

202 pages, 6x9 reprinting current ACI Standards: Building Code Requirements for Reinforced Concrete (ACI 318-47); Minimum Standard Requirements for Precast Concrete Floor Units (ACI 711-46); five recommended practices: Use of Metal Supports for Reinforcement (ACI 319-42); Measuring, Mixing and Placing Concrete (ACI 614-42); Design of Concrete Mixes (ACI 613-44); Construction of Concrete Farm Silos (ACI 714-46); Winter Concreting Methods (ACI 604-48); and two specifications: Concrete Pavements and Bases (ACI 617-44) and Cast Stone (ACI 704-44)—nine ACI Standards in one book, $2.00 per copy—to ACI Members, $1.25. The "Detailing Manual" (ACI 315-48) is available separately only at its established price—see above.

Air Entrainment in Concrete (1944)

92 pages of reports of laboratory data and field experience including a 31-page paper by H. F. Gonnerman, "Tests of Concretes Containing Air-entraining Portland Cements or Air-entraining Materials Added to Batch at Mixer," and 61 pages of the contributions of 15 participants in a 1944 ACI Convention Symposium, "Concretes Containing Air-entraining Agents," reprinted (in special covers) from the ACI JOURNAL for June, 1944, $1.25 per copy, 75 cents to members.

Air Entrainment in Concrete—Book 2 (1947)

A 204-page compilation of information on air entrainment published after Book 1 (1944), including: "Field Use of Cement Containing Vinsol Resin", and "Laboratory Studies of Concrete Containing Air-Entraining Admixtures", by Charles E. Wuerpel, "Entrained Air in Concrete", a foreword and 14 short papers presented at the 1946 Convention; and discussion of the symposium, reprinted from ACI JOURNALS for September 1945, Feb., June and Dec. Part 2, 1946. $2.25 per copy, $1.50 to ACI Members.

ACI Manual of Concrete Inspection (July 1941)

This 140-page book (pocket size) is the work of ACI Committee 611, Inspection of Concrete. It sets up what good practice requires of concrete inspectors and a background of information on the "why" of such good practice. Price $1.00—to ACI members 75 cents.

"The Joint Committee Report" (June 1940)

The Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete submitting "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete," represents the ten-year work of the third Joint Committee, consisting of affiliated committees of the American Concrete Institute, American Institute of Architects, American Railway Engineering Association, American Society of Civil Engineers, American Society for Testing Materials, Portland Cement Association. Published June 15, 1940, 140 pages. Price $1.50—to ACI members $1.00.

Reinforced Concrete Design Handbook (Dec. 1939)

This report of ACI Committee 317 is in increasing demand. From the Committee's Foreword: "One of the important objectives of the committee has been to prepare tables covering as large a range of unit stresses as may be met in general practice. A second and equally important aim has been to reduce the design of members under combined bending and axial load to the same simple form as is used in the solution of common flexural problems."—132 pages, price $2.00—$1.00 to ACI members.

For further information about ACI Membership and Publications (including pamphlets presenting synopses of recent ACI papers and reports) address:

AMERICAN CONCRETE INSTITUTE **New Center Building** **Detroit 2, Michigan**

given to the resistance of footings to failure by bond, diagonal tension and tension in the steel. Test footings were designed to produce these various types of failure. In addition, studies were made of the behavior of rectangular footings and footings having intermediate capitals or piers.

The tests featured measurements of tensile and compressive strains, deflections and slippage of bars. In some of the tests three grades of concrete were used, in others, five types of reinforcing bar were employed. In most of the tests, the footings were 7 ft square. They were supported on a bed of steel springs and tested in a large testing machine.

This paper merely attempts to present and interpret the test results, and does not offer recommendations as to possible changes in design procedures at this time.

LATTICE ANALOGY IN CONCRETE DESIGN.....45-7

Price 35 cents.

DOUGLAS McHENRY—Oct. 1948, pp. 129-140 (V. 45)

The lattice analogy is a scheme for solving two-dimensional stress problems in which the involved mathematical methods of the theory of elasticity are replaced by simple computations. The solid section is replaced by an equivalent lattice or framework which may be solved by methods applicable to indeterminate structures. Stress distribution in sections of complex shape and with complex loading may be determined by successive approximations which involve only substitution in simple equations. The method is illustrated by application to the problem of stresses in a deep beam with off-center loading.

STRENGTH OF PRECAST CONCRETE FLOOR JOISTS.....45-8

Pricd 35 cents.

JACOB FELD—Oct. 1948, pp. 141-148 (V. 45)

The strength and usability of precast concrete floor joists left in the open without any protection for a year, when a housing project was abandoned, were proved by load test on a slab section incorporating the poorest joists. This paper describes the condition of the joists, the load test results and indicates that a greater tolerance can be safely permitted in the visible defects of precast concrete joists.

EFFECT OF ENTRAINED AIR ON CONCRETES MADE WITH SO-CALLED "SAND-GRAVEL" AGGREGATES.....45-9

Price 35 cents.

PAUL KLIEGER—Oct. 1948, pp. 149-164 (V. 45)

Sand-gravel aggregates (maximum particle size $\frac{3}{8}$ in.) used with non-air-entraining cement produce concretes containing from 3 to 4 percent air. The same mixes made with air-entraining cement and the sand-gravel aggregates have air contents from 8 to 13 percent. Effect of this larger amount of entrained air is given in terms of tests of flexural and compressive strength, freezing and thawing resistance, and length changes in varying storage conditions. Effect of "sweetening" the sand-gravel by addition of 30 percent of coarser aggregate is noted.

BURNED SHALE AND EXPANDED SLAG CONCRETES WITH AND WITHOUT AIR-ENTRAINING ADMIXTURE.....45-10

Price 35 cents.

P. H. PETERSEN—Oct. 1948, pp. 165-176 (V. 45)

The physical properties of several lightweight aggregate portland cement concretes made with burned shale or expanded slag were investigated at the National Bureau of Standards. Three grades of concrete were made with each aggregate. Air-entrainment greater than 20 percent is reported for the mixtures leanest in cement, an air-entraining admixture being used to increase the workability of all but the richest concretes. Compressive, flexural and bond strength data are given as well as resistance to heat transfer, rain penetration and water penetration by capillarity. Also included are the coefficients of thermal expansion, shrinkage, and values for change in length due to wetting and drying.

CEMENSTONE PRECAST CONSTRUCTION.....45-11

Price 35 cents.

A. C. GRAFFLIN—Nov. 1948, pp. 193-204 (V. 45)

Standardized precast reinforced concrete structural members and panels point toward economy in fireproof construction of ordinary buildings up to four stories. The method described has been so developed that architects and engineers can select from tables reinforced concrete beams, columns, roof, floor, and wall panels in the same manner as for structural steel. Design is in accordance with the ACI Code and costs compare favorably with similar structures framed of steel and fireproofed with concrete.

SAND GRADING INFLUENCE ON AIR ENTRAINMENT IN CONCRETE.45-12

Price 35 cents.

M. A. CRAVEN—Nov. 1948, pp. 205-216 (V. 45)

Four series of mixes with varying air-entraining agents, cement factors, and sand grading and content were prepared in order to observe sand grading effect on concrete. Graphic record is presented of air content and W/C plotted against fineness modulus of sand; flow and compressive strength plotted against fineness modulus of sand; and air content plotted against percent of No. 30—No. 50 sand and total sand percentage. Generally the percentage of air entrained in concrete increased with decrease in fineness modulus of sand. Quantity of air appears to be a function of the quantity of No. 30—No. 50 sand. Effect of grading and quantity of sand on other properties of fresh and hardened concrete is noted.

INFLUENCE OF SIZE GRADING OF SAND ON AIR ENTRAINMENT.45-13

Price 35 cents.

E. W. SCRIPTURE, Jr., F. B. HORNIBROOK, and D. E. BRYANT—Nov. 1948, pp. 217-228 (V. 45)

Field reports indicated difficulty with certain sands in securing the desired amount of entrained air; this was frequently attributed to the size grading of the sand, particularly to a deficiency in the finer fractions. An experimental investigation was undertaken in view of the paucity of published data on this subject. Mixes of sand and water alone, 1:4 and 1:2 mortars, and concrete mixes were made with and without air-entraining agents and the air contents determined. While size grading of the sand had a very large influence on air entrainment in a mixture of sand and water alone, this effect was smaller in mortars and very small in concrete mixes. It was found that maximum air was entrained by the 28–48 mesh size sand rather than the 48–100 mesh size.

USE OF PREPACKED AGGREGATE CONCRETE IN MAJOR DAM CONSTRUCTION.....45-14

Price 35 cents.

PAUL BAUMANN—Nov. 1948, pp. 229-236 (V. 45)

This paper sets forth the method of construction of a major gravity dam whereby the average temperature of the mass concrete may be controlled within the mean annual and the dry shrinkage reduced to a fraction of that of conventional concrete by means of prepacking of the coarse aggregate to a density corresponding to 30 percent voids; precooling the aggregate; the intrusion of the coarse aggregate by mortar with a cement content of two sacks per cu yd or less; and the internal post-cooling, curing and draining of the mass concrete. (This process is known as the Prepakt method of construction.)

Continued on p. 22

Current ACI Standards

Building Code Requirements for Reinforced Concrete (ACI 318-47)
64 pages in covers: 50 cents per copy (40 cents to ACI Members)

Minimum Standard Requirements for Precast Concrete Floor Units (ACI 711-46)
16 pages in covers: 50 cents per copy (40 cents to ACI Members)

Recommended Practice for the Construction of Concrete Farm Silos (ACI 714-46)
16 pages in covers: 50 cents per copy (40 cents to ACI Members)

Recommended Practice for the Design of Concrete Mixes (ACI 613-44)
24 pages in covers: 50 cents per copy (40 cents to ACI Members)

Specifications for Cast Stone (ACI 704-44)
4 pages: 35 cents per copy

Specifications for Concrete Pavements and Bases (ACI 617-44)
30 pages in covers: 50 cents per copy (40 cents to ACI Members)

Recommended Practice for Measuring, Mixing and Placing Concrete (ACI 614-42)
30 pages in covers: 50 cents per copy (40 cents to ACI Members)

Recommended Practice for the Use of Metal Supports for Reinforcement (ACI 319-42)
4 pages: 35 cents per copy

Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-48)
A publication of large format, bound to lie flat.
55 pages: $3.00 per copy ($1.75 to ACI Members)

Recommended Practice for Winter Concreting Methods (ACI 604-48)
20 pages in covers: 50 cents per copy (40 cents to ACI Members)

Recent Proposed ACI Standards

Proposed Recommended Stresses for Unreinforced Concrete
Reported by Committee 322 as information and for discussion only. 4 pages, 35 cents per copy. (Reprint from ACI JOURNAL, Nov. 1942)

Proposed Recommended Practice for the Application of Portland Cement Paint to Concrete Surfaces
Released by Committee 616 and the Standards Committee for publication for consideration for adoption as a standard. 16 pages, 35 cents per copy (Reprint from ACI JOURNAL, Jan. 1949)

RATIONAL ANALYSIS AND DESIGN OF TWO-WAY CONCRETE SLABS.....45-15

Price 60 cents.

C. P. SIESS and N. M. NEWMARK—Dec. 1948, pp. 272-316 (V. 45)

A new method for the design of two-way building slabs is proposed and its step-by-step development is described. A new moment distribution procedure is used to compute moments in a number of rectangular slabs continuous over rigid beams. Several variables are studied in the analyses and include: the ratio of sides, the effect of discontinuous edges, the torsional stiffness of the beam, various types of loading, and combinations of panels of various sizes and shapes. Certain conclusions are stated regarding the types of loading to be considered and the values of beam torsional stiffness to be assumed in the development of the design procedure.

Moments obtained in the foregoing analyses are modified to take into account the effect of additional variables. First, the slab moments are increased by various amounts as a result of the deflection of the beams. Next, separate coefficients for dead and live load are replaced by combined coefficients for a live load-dead load ratio of 3.0. Then follows a study to determine the effect of discontinuous edges, and this variable is eliminated from the procedure. Finally all slab moments are reduced by 20 percent in recognition of the redistribution of moments at high load. Additional studies are then made of the distribution of moments across the width of the slab and of the moments to be used for the design of the beams to complete the development of the design procedure.

A proposed design procedure is presented in specification form and is compared with other procedures of a similar nature and with the moment distribution procedure described in the appendix.

CORROSION RESISTANCE TESTS OF CONCRETE FLOORS—WITH AND WITHOUT METALLIC AGGREGATE.....45-16

Price 35 cents.

E. W. SCRIPTURE, JR. and C. H. SAKRYD—Dec. 1948, pp. 317-324 (V. 45)

An attempt has been made to determine the conditions under which concrete floors can be expected to give good service. Resistance to abrasion was determined on specimens with and without metallic aggregate after exposure to typical reagents selected from the groups described by the Joint Committee as having corrosive effects of an intermediate nature between no attack and severe attack. On the basis of the experimental results the reagents in the Joint Committee report have been classified in relation to anticipated service of concrete floors.

FLEXICORE PRECAST FLOOR AND ROOF SLABS.....45-17

Price 35 cents.

GAYLE B. PRICE—Dec. 1948, pp. 325-340 (V. 45).

This paper describes briefly the design, manufacture, erection procedures and use of Flexicore slabs in precast concrete floor and roof systems. The Flexicore slab has a standard cross section 12 in. wide and 6 in. thick, lightened by two holes 4⅝ in. in diameter running the full length of the slab which is made in inch variations of length up to a maximum of 22 ft 6 in. Reinforcement consists of two bars in the top of the slab for "handling" stresses and two or three prestressed tension bars at the bottom to take care of bending moment.

PROPOSED RECOMMENDED PRACTICE FOR THE APPLICATION OF PORTLAND CEMENT PAINT TO CONCRETE SURFACES.....45-18

Price 35 cents.

REPORT of COMMITTEE 616—Jan. 1949, pp. 353-368, (V. 45)

This report is a studied revision by a reorganized committee of the previous report of Committee 616 published in June 1942, based upon criticisms of that report plus information from more recent tests and experiences. Included are recommendations for appropriate usage, age of concrete, preparation of surface, and the preparation, application and curing of portland cement paint. Three appendixes discuss composition, manufacture and storage and, general characteristics and factors affecting durability.

DRY MORTAR AS A BEARING AND GROUTING MATERIAL.....45-19

Price 35 cents.

BOYD S. BROOKS—Jan. 1949, pp. 369-380, (V. 45)

In 1939, at the navy's new David Taylor Model Basin at Carderock, Md., there was an urgent need for a non-shrinking bearing material to be used under the cast-steel "chairs" which support the rails on which the towing carriages run. Requests for information concerning materials and installation procedures produced such varied answers that investigation was started to find the best material and the best procedures for placing it.

The bearing material selected for use was 1:3 portland cement mortar, with 4.13 gallons of water per sack of cement. This was rammed firmly into place.

This bearing material has now been in place over a period of about 9 years under extremely severe conditions and without exception has proved to be excellent in function and condition.

DIRECT DIMENSIONING OF RECTANGULAR SECTIONS.....45-20

Price 35 cents.

MICHEL BAKHOUM—Jan. 1949, pp. 381-396 (V. 45)

On the basis of the standard theory for reinforced concrete design, methods have been derived for the direct dimensioning of rectangular sections, with or without compression reinforcement, when subjected to simple bending, eccentric compression or eccentric tension. Two cases are considered: when the depth of the section is fixed and when it is not fixed. The conditions leading to minimum reinforcement are also studied. The solutions developed have been simplified by the use of curves which are applicable for pure bending as well as for eccentric compressive or tensile forces and cover any values o f_c, f_s and n. The curves allow making a variety of designs in a relatively short time and thus help in choosing the most economical or most suitable section.

STRENGTHENING BRIDGE SLABS WITH GROUTED REINFORCEMENT.45-21

Price 35 cents.

S. O. ASPLUND—Jan. 1949, pp. 397-408

On the project described the negative reinforcing bars settled as much as 2½ in. out of correct position. Various means for incorporating negative reinforcement at correct height in the finished structure are discussed. The method selected of grouting additional bars in grooves cut with the aid of diamond saw is described. Tests on beam specimens made with some bars encased in the concrete and some bars grouted into diamond-sawed grooves, give results identical with those to be expected for all bars normally encased; ultimate strengths conform closely to the plastic bending capacity of reinforced concrete according to design methods originated by C. S. Whitney.

COMPARATIVE DESIGNS OF A SEGMENTAL SKEWED FRAME CONCRETE BRIDGE BY THE STRAIGHT LINE AND PLASTIC THEORY METHODS.....45-22

Price 35 cents.

MILTON BRUMER—Jan. 1949, pp. 409-420 (V. 45)

The designs of a segmental skewed frame concrete bridge are discussed and compared as developed by both the straight line and plastic theory methods. The plastic theory will lead to rigid concrete frames of more economical proportions capable of sustaining equal or greater loads than frames designed by the straight line method. Effects of volume changes and yielding supports are also of smaller consequence. Deflections may be expected to be greater. The author concludes that further substantiation of the plastic theory is desirable.

Tools, Materials, Services

Under this heading note will be made from time to time of producer literature of presumed technical interest (and available from its source for the asking) to ACI users of tools, equipment, materials, accessories and special services.

Wall form construction

Only three essential pieces of hardware—form tie, short connecting bolt and tightening wedge—comprise the basis of the Symons system of wall form construction described in a recent folder issued by Symons Clamp and Manufacturing Co., 4277 West Diversey Avenue, Chicago. This publication also shows details of walers and waler fastenings and indicates the scope of free engineering service provided by the company.

Grooving machine for concrete

A gasoline engine powered unit that cleans out sealing material, dirt and foreign matter and cuts joints so that new, rough concrete surfaces are provided on each side wall of the joint is described in literature from the G. H. Tennant Co., 2530 N. Second Street, Minneapolis 11, Minn. Self propelled and requiring a single operator, the machine is said to be capable of cutting and cleaning both straight and curvilineal joints to a depth not less than ¾ in., and a width varying from ½ to 2½ in. Performance data are available from the manufacturer.

Hard facing on mixer arms

Recently received from Wall Colmonoy Corp., 19345 John R Street, Detroit, Mich., a 3-page booklet describes the hard facing of mixer arms on concrete and cinder block equipment. It gives information on the method of welding an overlay of Colmonoy to leading edge and outside faces of mixer arms, with sketches showing exactly where to place the hard facing. Method is said to increase life of equipment four to five times.

Practical concrete course

The third edition of "A Practical Course in Concrete", by Henry Giese, has been announced by the publisher, Portland Cement Association, 33 West Grand Avenue, Chicago 10. This booklet has been completely revised to serve as a teaching guide and laboratory manual for instructors giving courses in concrete work at agricultural colleges and vocational agricultural high schools. The manual will also serve for special courses, such as those given to veterans. It can also be used as a textbook in which study assignments can be made. District offices of the association will offer this manual to college, vocational training and high school instructors giving courses in concrete work.

Available only in U. S. and Canada.

Mix design and control

The Portland Cement Association has also released the ninth edition of "Design and Control of Concrete Mixtures". The revision of this popular booklet provides up-to-date information on design of mixtures, construction practices and control tests. More information is given on types of cement, requirements for durability and methods of handling materials and concrete than in former editions. Suggestions for designing mixtures for air-entraining concrete have been included.

Available only in U. S. and Canada.

Calcium chloride

The use of calcium chloride in portland cement concrete is described in a booklet available from Solvay Sales Corporation, 40 Rector Street, New York 6. Its five sections are devoted to facts showing the results produced by the addition of calcium chloride to portland cement mixes, practical applications, answers to questions most frequently asked by construction people, abstracts from and references to technical papers and reports supporting the facts presented in the first section, and suggested specifications and recommendations on the use of calcium chloride.

Publications of The National Bureau of Standards

Circular 460, *Publications of the National Bureau of Standards*, listing all Bureau publications from 1901 to June 30, 1947, is now available from the Superintendent of Documents.

Bureau publications listed in the circular include mathematical tables, building materials and structures reports, circulars, handbooks, commercial standards, simplified practice recommendations, research papers, and miscellaneous publications. Brief abstracts for the publications issued from January 1, 1942, to June 30, 1947, are also included. Circular 460, 375 pages, can be obtained from the Superintendent of Documents, U. S. Government Printing office, Washington 25, D. C., price 75 cents.

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Discussion closed January 1, 1949

Sept. Jl. '48

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Relative Strengths of Portland Cement Mortar in Bending Under Various Loading Conditions—Jacob Porter Frankel

Terrazzo as Affected by Cleaning Materials—D. W. Kessler

Transporting Ready-Mixed Concrete in Open Dump Trucks—R. A. Burmeister

A Study of Alkali-Aggregate Reactivity by Means of Mortar Bar Expansions—T. M. Kelly, L. Schuman, F. B. Hornibrook

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Strength of Precast Concrete Floor Joists—Jacob Feld

Effect of Entrained Air on Concretes Made With So-Called "Sand-Gravel" Aggregates—Paul Klieger

Burned Shale and Expanded Slag Concretes With and Without Air-Entraining Admixture—P. H. Petersen

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Reinforced Concrete Wall and Column Footings, Parts 1 and 2—Frank E. Richart

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Cemenstone Precast Construction—A. C. Grafflin

Sand Grading Influence on Air Entrainment in Concrete—M. A. Craven

Influence of Size Grading of Sand on Air Entrainment—E. W. Scripture, Jr., F. B. Hornibrook and D. E. Bryant

Use of Prepacked Aggregate Concrete in Major Dam Construction—Paul Baumann

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Corrosion Resistance Tests of Concrete Floors—With and Without Metallic Aggregate—E. W. Scripture, Jr. and C. H. Sakryd

Flexicore Floor and Roof Slabs—Gayle B. Price

Discussion closes May 1, 1949

Jan. Jl. '49

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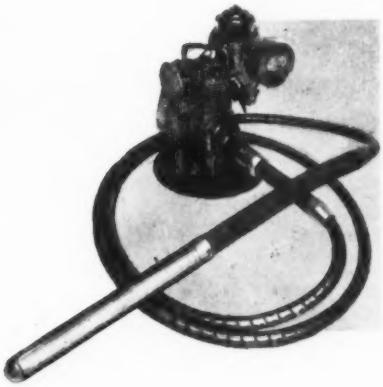

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JOURNAL
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AMERICAN CONCRETE INSTITUTE
(ACI PROCEEDINGS Vol. 45)

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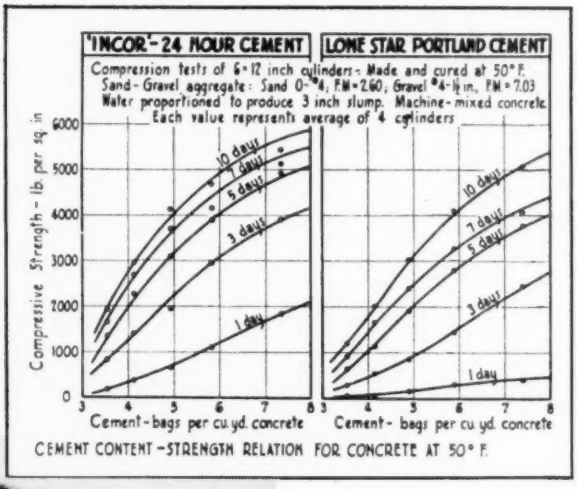
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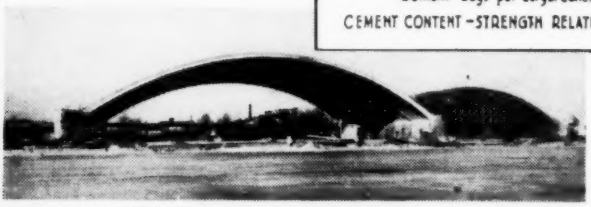

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Vol. 20—No. 6 February 1949 (Proceedings Vol. 45)

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Papers, committee reports and other contributions to the **Proceedings** pages of this Journal, as submitted in furtherance of Institute objectives, are, if published, accepted on the sole responsibility of their authors. Institute authority attaches only to standards adopted as provided in the By-Laws. All manuscripts prepared with a view to Journal publication should be submitted in triplicate, addressed: Secretary, Technical Activities Committee, American Concrete Institute, New Center Building, Detroit 2, Michigan.

Advertising—See statement under "Alphabetical List of Advertisers", final page **News Letter** section.

Journal subscription price $10.00 per year payable in advance. Members receive the Journal as one of the considerations of dues payment. The Journal is issued monthly, January to June and September to December (the volume year September to June being completed by title page, contents, indexes and closing discussion in Part 2 of the subsequent December issue).

Bound volumes 1 to 43 of PROCEEDINGS of the AMERICAN CONCRETE INSTITUTE (1905 to 1947) are for sale as far as available, at prices to be had on inquiry of the Secretary-Treasurer. Special prices apply for members ordering bound volumes in addition to the monthly Journal.

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This page of a pre-convention Exhibit issue of the ACI JOURNAL is a bow to the engineering and industry upon which the advanced students of good concrete depend to implement their new knowledge—to transform that knowledge from the static to the dynamic—"to make it work."

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The most academic of theoreticians are dependent on those of the ACI who devise, design and process the tools, machinery and materials to put into the hands of construction superintendents on jobs where good concrete is to be made.—Editor.

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POLICY

It is the policy of the American Concrete Institute to encourage participation by its members and others in the work of extending the knowledge of concrete and reinforced concrete as a basis for improved products and structures.

To this end the Board of Direction has assigned to the Technical Activities Committee the responsibility of selecting for publication such papers, committee reports, discussions and other contributions or parts of such contributions as, in the judgment of the Committee, seem to offer most of value in attaining Institute objectives within space requirements, consistent with budget limitations.

Read "Who ARE and What IS the American Concrete Institute," for an insight into the fields of interest and activity of ACI members, based on a sampling of classification returns. Are Institute activities and aims consistent with these interests? Turn to News Letter, p. 3, for a preliminary analysis of this important question.

Two convention papers, one on air-entraining concrete and one on design methods, appear in this issue:

R. F. Blanks and W. A. Cordon give Bureau of Reclamation "Practices, Experiences, and Tests with Air-Entraining Concrete," (p. 469).

In "Experimental Aids in Structural Design," (p. 445) R. E. Glover, O. J. Olsen, and Carl Zanger describe equipment useful in the solution of indeterminate structures.

Pre-convention publication offers an opportunity to prepare enlivening discussion questions.

"Manual of Standard Practice for Detailing Reinforced Concrete Structures," is again available. For the third printing (February, 1949) the price remains at $3.00, to ACI members $1.75.

Discussion of papers and reports in the October JOURNAL closed February 1, 1949. For papers in this issue, discussion in triplicate should be addressed to Secretary, ACI Technical Activities Committee, New Center Building, Detroit 2, Michigan. Closing date, June 1, 1949.

Manuscripts of papers, discussions and reports should be sent in triplicate to:

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In ACI Journal next month

March JOURNAL pages will be devoted to four papers to be delivered by their authors at the 45th annual convention in New York. Two solutions to highway pavement scaling problems are offered: oil treatment of existing pavements, and use of air entraining concrete in new construction. Architects, engineers and contractors will find useful pointers on architectural concrete in the other two papers. Further details in the summaries below.

B. D. TALLAMY, in "Control of Salt Scaling of Concrete Pavements," tells how a field observation of reduced scaling of New York highways on which oil had dripped from passing vehicles was verified by extensive laboratory tests; leading to the development of an oil-treating mixture and the necessary equipment to apply it to existing pavements. Highways so treated are expected to withstand straight salt treatment without appreciable scaling.

In "New Jersey Experience with Air-Entraining Concrete," CHARLES N. NOBLE reviews State Highway Department use of air-entraining concrete since 1945 and reports on condition of highways in which it was used. The continued use of concentrated chemicals for ice removal has led to general use of air-entraining concrete wherever contact with chemicals is expected. He includes specifications and field methods devised for adequate control of air entrainment.

J. J. HOGAN emphasizes the need for greater coordination of design with construction practices and limitations in "Design Details for Architectural Concrete." Form materials, ornamentation, construction and control joints, and reinforcement are discussed in this light. He suggests how desired results may be obtained at minimum cost with freedom from unwanted effects.

E. B. OBERLY looks at problems of architectural concrete from the contractor's viewpoint in "Construction Practices for Architectural Concrete." Despite rigid and detailed specifications the contractor still must plan and build forms properly to prevent undesirable leakage or bulging. He illustrates ways to lock forms, provide smooth surfaces and acceptable ornamentation, and produce good joints. He discusses concrete quality, form stripping and surface patching and finishing.

From tests on concrete mixes under standardized conditions which allowed only the factor under study to vary, the authors conclude that the amount of air entrained in concrete decreases with increasing cement factor and increases, generally, with slump and sand-total aggregate ratio.

Some Factors Affecting Air Entrainment*

By E. W. SCRIPTURE, Jr.,† and F. J. LITWINOWICZ‡

SYNOPSIS

A program was undertaken to investigate the influence of various factors on the amount of air entrained in concrete mixes with and without air-entraining agent added. The variables so far studied are slump, cement factor, and sand-total aggregate ratio. The results indicate that the two latter factors are of major importance but that slump has less effect. In general, entrained air content increases with increasing slump, decreases with increasing cement factor, and increases with increasing sand-total aggregate ratio

INTRODUCTION

In view of the importance of controlling the amount of entrained air in air entraining concrete, a study was undertaken of some of the variables which influence this quantity. There are a large number of such factors which affect, or are thought to affect, the amount of entrained air. These include the nature of the air-entraining agent itself, the proportion of air-entraining agent, the manner in which it is used, mix design, sand-total aggregate ratio, mixing time, type of mixer, grading of the aggregates, temperature, slump, cement factor, inherent characteristics of the cement, and probably others.

While numerous data have been published[1] on one or more of these variables in studies made for that purpose or incidental to other work, there appears to be a lack of data giving direct comparisons devoted solely to the evaluation of air contents. Therefore, a program was undertaken to determine the relations between each factor and air content by direct comparisons confined to a single variable. It has not been the purpose of this work to attempt to evaluate specific air-entraining

*Received by the Institute July 6, 1948. Title No. 45-23 is a part of the copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 6, Feb. 1949, *Proceedings* V. 45. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than June 1, 1949.

†Member American Concrete Institute, Vice President in charge of research, The Master Builders Co. Cleveland, Ohio.

‡Member American Concrete Institute, senior research assistant, physical testing section, research laboratory, The Master Builders Co., Cleveland, Ohio.

[1]For a bibliography of 88 references bringing this subject up to 1945 see Wuerpel, Charles E., "Laboratory studies of Concrete Containing Air-Entraining Admixtures," ACI JOURNAL, Feb. 1946, *Proc.* V. 42 p. 305. Additional references to June 1947 are found in the ACI 10-Year Index. A more recent reference is Meissner, H. S., "Vibration of Air-Entrained Concrete," ACI JOURNAL Oct. 1947, *Proc.* V. 44 p. 182.

agents, the proportions in which they are to be used to produce a given effect, or their manner of use, but to evaluate other factors under a given set of conditions. For this purpose comparisons have been made with the three air-entraining agents now in common use, employed in proportions commonly used in practice, and added at the mixer to avoid any complications which might be involved in intergrinding.

The first variable investigated was the grading of the sand, as this seemed to be a factor which has been considered of paramount importance with respect to air entrainment. The results of this investigation have been reported in a previous paper.[2] Work on three additional variables, slump, cement factor, and sand-total aggregate ratio, has now been completed and is the subject of this paper.

DESCRIPTION OF TESTS

In general, four series of tests were made, one with no air-entraining agent and three with each of three different air-entraining agents added, namely, Admixture A—petroleum hydrocarbon insoluble fraction of a coal-tar hydrocarbon extract of pine wood, Admixture B—triethanolamine salt of a sulphonated hydrocarbon, and Admixture C—a calcium lignosulphonate cement dispersing agent. In each case also, mixes were made which were both rich and lean. The amounts of Admixture A and B were selected to give between 4 and 5 percent of air, with a 5½-sack mix at about a 3-in. slump. The amount of Admixture C used was that recommended by the manufacturer, ½ lb per sack of cement.

In the investigation of the effect of slump on entrained air, slumps were varied between approximately ½ in. and 9 in. Two sets of tests were made, one with a nominal 6-sack mix to represent rich mixes, and one with a nominal 4-sack mix to represent lean mixes.

In investigating the effect of cement factor on entrained air, the cement factor was varied from about 3½-sacks per cu yd, which is somewhat lower than is encountered in general practice, to about 7-sacks per cu yd, which is about as high as is commonly used. The sand-total aggregate ratio varied from .52 to .38 from the leanest to the richest mix. The slump throughout this series was maintained as near 3 in. as possible. To explore more carefully the variations in entrained air in richer mixes, a second series was run, varying the cement factor in small increments from 5½-sacks to 7-sacks per cu yd.

For the study of the effect of sand-total aggregate ratio on entrained air, mixes with three cement factors, 4½, 5½, and 6½ sacks per cu yd, were used. The slump was maintained at approximately 3½ in. The

[2]Scripture, E. W. Jr., Hornibrook, F. B. and Bryant, D. E., "The Influence of the Size Grading of Sand on Air Entrainment," ACI JOURNAL, Nov. 1948, *Proc.* V. 45, pp. 217-228.

sand-total aggregate ratios covered mixes from relatively harsh and stony to definitely over-sanded, in each series. Numerically, these ratios by solid volumes varied from .32 to .56, but in each series due allowance was made for the richness of the mix, so that the ranges were .32 to .48, .36 to .52, and .40 to .56 for the 6½, 5½, and 4½-sack mixes, respectively.

The same brand of cement used throughout the work was a normal portland cement, Type I, with a surface area of 1725 sq cm/g giving 7.0 percent air in the ASTM mortar test C185-47T. The sand was local bank sand and the coarse aggregate was crushed limestone having size gradings given in Table 1.

TABLE 1—SIEVE ANALYSIS OF SAND AND AGGREGATE

Sand			Aggregate		
Sieve size	Percent retained	Cumulative percent	Sieve size	Percent retained	Cumulative percent
4 mesh	4	4	1½ in. screen	0	0
8 mesh	17	21	1 in. screen	54	54
14 mesh	19	40	¾ in. screen	33	87
28 mesh	21	61	½ in. screen	11	98
48 mesh	30	91	⅜ in. screen	2	100
100 mesh	8	99	4 mesh	0	
Pan	1	100			

The concrete mixes were made in batches of approximately 2 cu ft, mixed 3 minutes in a 2½ cu ft tilting-drum type concrete mixer. Slumps were determined by standard method and the air contents were determined immediately after mixing with a Klein-Walker air meter.

Relation of air content to variable slump

In the series on variable slump, the variations in air content (Fig. 1 and 2 and Table 2) with no air-entraining agent added are very small, and it may be doubted whether they are of any significance. In the higher slump range, with the 6-sack mix, there is some indication that the air contents are decreased, possibly through the escape of air from a rather fluid mix.

With the air-entraining agents there do not appear to be any definite variations in air content with slump, but with the two agents giving air contents in the higher range there seems to be some indication of a general tendency for air content to increase with increasing slump. With Admixtures A and B a peak in air content appears to be reached at about a 5-in. slump but the differences are hardly large enough to warrant a definite conclusion. This is in accord with data published by Gonnerman[3] in his investigation of air entrainment, but his data were secured

[3]Gonnerman, H. F., "Tests of Concretes Containing Air-Entraining Portland Cements or Air-Entraining Materials Added to Batch at Mixer," ACI JOURNAL June 1949, *Proc.* V. 40, pp. 477-508.

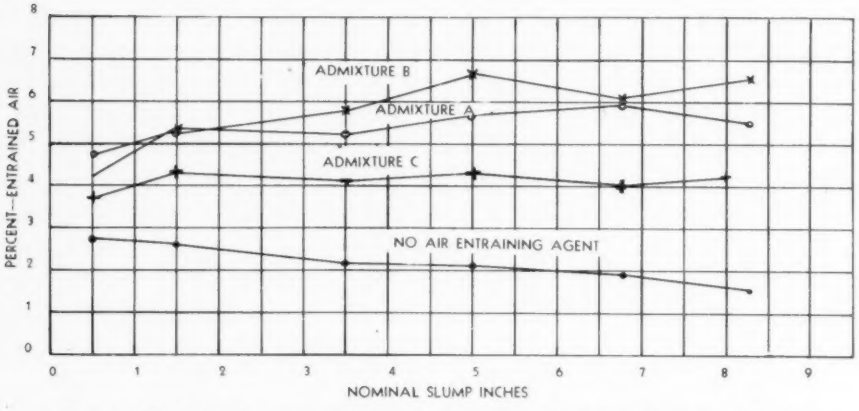

Fig. 1—Variation in entrained air with slump—nominal 6-sacks per cu yd mix

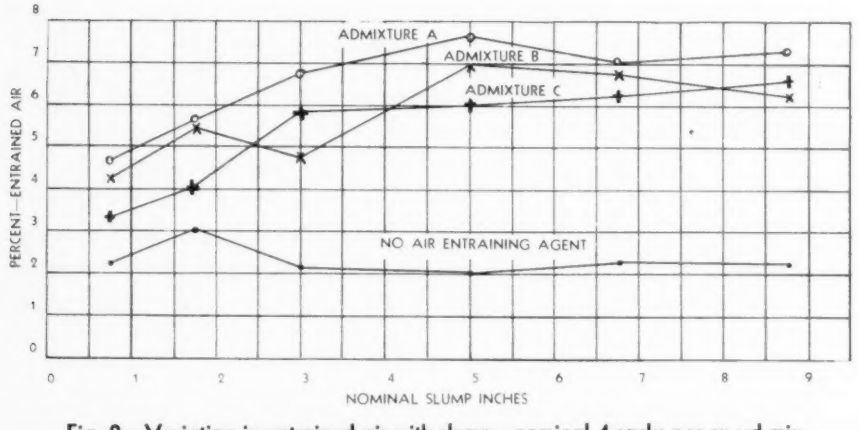

Fig. 2—Variation in entrained air with slump—nominal 4-sacks per cu yd mix

for another purpose and he has not made direct comparison of air content as related to slump. Apparently with this type of mix, even with slumps as high as 8 to 9 in., the mixes are still sufficiently cohesive to prevent any substantial escape of air in the interval between mixing and determination of the air content. With mixes with less than 1 in. slump, all cases show definitely lower air contents than mixes over the remaining range of slumps and there is some indication that less air is entrained in the drier mixes with somewhat higher slumps. It seems probable that this is due to the difficulty of beating air into a very stiff mix, and would appear to be related more to the effectiveness of the mixing action than to any inherent effect of consistency on air content. No significance should be attached to the relative air contents secured with the three air-entraining agents, since two of these were used in arbitrarily

TABLE 2—AIR CONTENT IN RELATION TO WATER-CEMENT RATIO AT VARYING SLUMP

Nominal slump, in.	Air-entraining agent added							
	None		Admixture A		Admixture B		Admixture C	
	Percent entrained air	W/C, gal. per sack	Percent entrained air	W/C, gal. per sack	Percent entrained air	W/C, gal. per sack	Percent entrained air	W/C, gal. per sack
Nominal 6-sack mix								
½	2.8	4.48	4.7	4.57	4.2	4.68	3.7	4.18
1½	2.7	4.95	5.5	4.98	5.4	4.78	4.3	4.49
3½	2.2	5.34	5.3	5.30	5.8	5.09	4.1	4.95
5	2.1	5.57	5.8	5.44	6.7	5.46	4.3	5.16
6¾	1.9	5.80	6.0	5.58	6.0	5.55	4.0	5.35
8¼	1.6	6.00	5.6	6.00	6.5	5.95	4.2	5.51
Nominal 4-sack mix								
¾	2.3	7.45	4.6	6.72	4.3	7.12	3.4	7.05
1¾	3.1	8.04	5.6	7.11	5.5	7.36	4.1	7.35
3	2.2	8.17	6.8	7.45	4.7	7.65	5.9	7.77
5	2.2	8.71	7.7	7.86	7.0	8.31	6.1	8.11
6¾	2.3	8.97	7.1	8.21	6.8	8.66	6.3	8.48
8¾	2.3	9.06	7.4	8.71	6.3	8.91	6.7	8.71

selected proportions to give air contents in a certain range, and this range could have been changed by altering the amount of air-entraining agent used.

Cement factor

As seen in Fig. 3 and Table 3, the air content decreases regularly with increase in cement factor. This is true for the series without an air-entraining agent, as well as for the three series with air-entraining agents, but is at a smaller rate in the former case. Over the range 3½-sacks to 5½-sacks per cu yd, with Admixture C, there do not appear to be any significant differences in air content. With Admixture B there is practically no change in air content between 6¼ and 7-sacks per cu yd. With these minor exceptions the results as a whole are remarkably consistent in indicating the decrease in air content with increase in cement factor. This is consistent with Gonnerman's data[3], but in both these data and the results reported here, the sand-total aggregate ratio necessarily was varied simultaneously with the cement factor, so that there might be some question whether the observed differences in air content are attributable to the changes in cement factor or in sand-total aggregate ratio, or in both.

For the series covering cement factors between 5½ and 7-sacks per cu yd (Fig. 4), Admixtures A and B show the general tendency toward

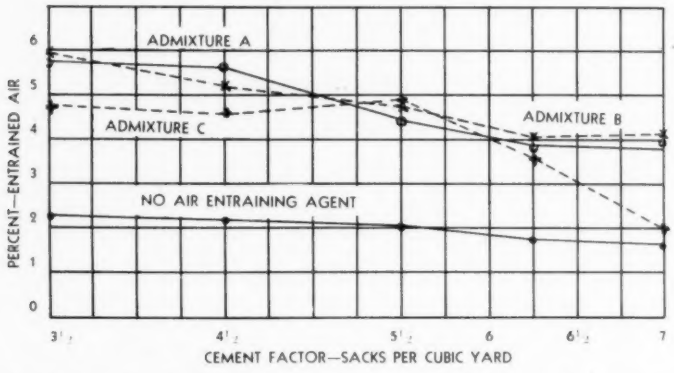

Fig. 3—Variation in entrained air with cement factor—3½ to 7 sacks per cu yd

TABLE 3—AIR CONTENT AND WATER-CEMENT RATIOS AT VARIABLE CEMENT FACTORS—NOMINAL SLUMP 3½ IN.

Nominal cement factor	Air-entraining agent added							
	None		Admixture A		Admixture B		Admixture C	
Sacks per cu yd	Percent entrained air	*W/C*, gal. per sack	Percent entrained air	*W/C*, gal. per sack	Percent entrained air	W/*C*, gal. per sack	Percent entrained air	*W/C*, gal. per sack
3½	2.3	10.25	5.8	9.20	5.9	9.42	4.8	8.75
4½	2.2	7.31	5.6	6.58	5.2	6.97	4.6	6.44
5½	2.0	5.79	4.4	5.55	4.8	5.65	4.9	5.29
6¼	1.7	5.36	3.9	4.92	3.9	5.11	3.5	4.65
7	1.6	4.51	3.7	4.80	4.0	4.71	2.0	4.29
5½	1.4	5.83	5.3	5.50	5.2	5.60	2.5	5.36
6	1.1	5.43	5.0	5.04	5.2	5.09	2.8	4.98
6¼	1.3	5.10	4.6	4.99	5.2	5.11	2.8	4.66
6½	1.4	4.98	4.6	4.86	4.6	4.97	2.7	4.63
6¾	1.2	4.76	4.3	4.83	4.7	4.85	2.5	4.46
7	1.2	4.65	4.3	4.69	4.5	4.67	2.3	4.37

decrease in air with increase in cement factor, but with Admixture C and with no air-entraining agent the differences found in this range are hardly of a magnitude to be significant.

Sand-total aggregate ratio

The results in the series to investigate the influence of sand-total aggregate ratio on entrained air with no air-entraining agent show (Fig. 5 and Table 4) that entrained air increases with increasing sand-total aggregate ratio. There is some indication that above a certain maximum, as shown by the points for the 5½ and 6½-sack mixes, the amount of entrained air is not further increased. It may be noted that the leaner mix, 4½-sacks per cu yd, shows lower air contents than the

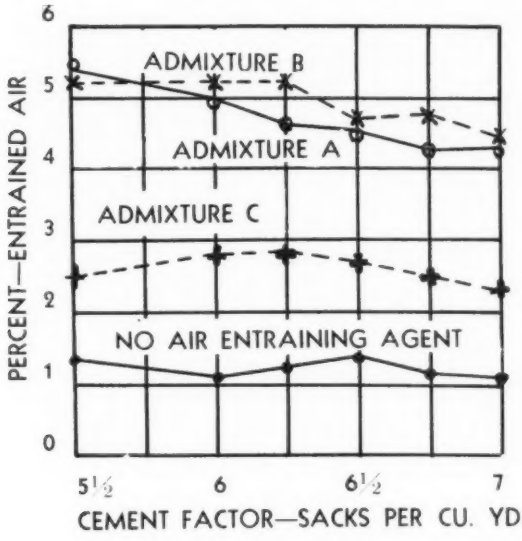

Fig. 4—Variation in entrained air with cement factor—5½ to 7 sacks per cu yd

two richer mixes, 5½ and 6½-sacks per cu yd, which is contrary to the previous general observation that air content decreases with increasing cement factor. This is, however, consistent with the previous observation that with no air-entraining agent added the differences in air content with varying richness of mix were not very significant.

With the three air-entraining agents the situation is similar, in that air content increases fairly regularly with increasing sand-total aggregate ratio. The data from Gonnerman's work[3] lead to the same conclusion but his mixes varied simultaneously in sand-total aggregate ratio and in cement factor, whereas the series here reported were each made with a fixed cement factor varying only the sand-total aggregate ratio. There is some indication that with Admixtures A and B in the leanest mix, 4½-sacks per cu yd, which are those mixes in which the highest air contents were secured, no further increase is produced by increasing the sand-total aggregate ratio after a certain maximum air content is reached. In fact, some decrease appears to occur.

It might be thought that the increase in air content with increasing sand-total aggregate ratio was merely the result of increasing the percentage of mortar in the mix and that the air content of the mortar remained constant. This being true, the results expressed as percentage air in the mortar would make the curves in Figs. 5, 6, 7, and 8 all become straight lines parallel with the horizontal axis. This is not the case and, in fact, recalculation of the entrained air on this basis does not significantly affect any of the relations in the study of the influence of sand-total aggregate ratio.

TABLE 4—AIR CONTENT AND WATER CEMENT RATIOS WITH VARIABLE SAND-TOTAL AGGREGATE RATIOS—NOMINAL SLUMP 3½ IN.

A-E agent	Cement factor sacks	Sand-total aggregate ratio													
		.32		.36		.40		.44		.48		.52		.56	
		Percent entrained air	W/C, gal. per sack	Percent entrained air	W/C, gal. per sack	Percent entrained air	W/C, gal. per sack	Percent entrained air	W/C, gal. per sack	Percent entrained air	W/C, gal. per sack	Percent entrained air	W/C, gal. per sack	Percent entrained air	W/C, gal. per sack
None	4½	–	–	–	–	0.8	6.64	1.2	6.83	1.4	7.19	1.7	7.60	2.0	8.20
None	5½	–	–	0.8	5.59	2.0	5.64	3.2	5.76	3.1	5.85	3.1	6.04	–	–
None	6½	0.6	4.85	1.9	4.97	2.0	5.05	3.2	4.85	3.1	4.88	–	–	–	–
A	4½	–	–	–	–	7.2	6.24	8.1	6.41	9.0	6.79	8.4	6.91	7.5	7.11
A	5½	–	–	4.8	5.31	5.3	5.35	6.3	5.26	6.5	5.52	7.2	5.62	–	–
A	6½	4.2	5.03	4.9	4.90	5.6	4.96	5.2	4.90	6.0	5.03	–	–	–	–
B	4½	–	–	–	–	4.0	6.49	6.0	6.66	7.7	6.61	7.6	6.79	7.1	7.02
B	5½	–	–	4.5	5.30	5.0	5.45	5.5	5.51	5.9	5.54	6.5	5.65	–	–
B	6½	3.8	4.95	4.6	4.85	5.1	4.66	5.1	4.75	5.8	4.96	–	–	–	–
C	4½	–	–	–	–	3.9	5.89	4.1	6.04	4.3	6.32	5.1	6.56	6.7	6.65
C	5½	–	–	2.0	5.04	2.9	5.02	3.3	5.05	4.0	5.22	4.0	5.45	–	–
C	6½	2.0	4.42	2.2	4.50	2.5	4.60	3.0	4.66	3.7	4.80	–	–	–	–

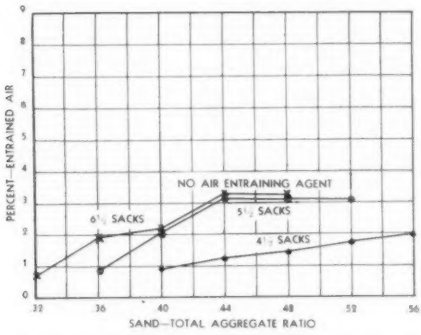

Fig. 5 (left)—Variation in entrained air with sand-total aggregate ratio—no air entraining agent

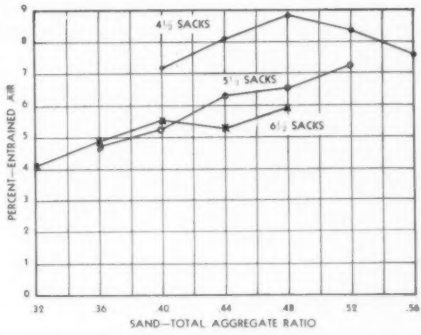

Fig. 6 (right)—Variation in entrained air with sand-total aggregate ratio—admixture A

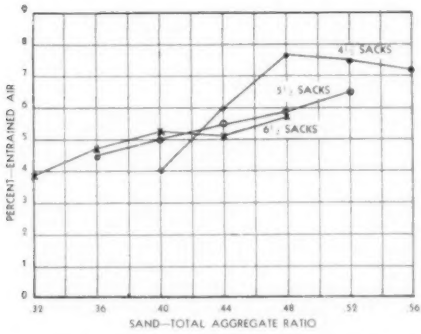

Fig. 7 (left)—Variation in entrained air with sand-total aggregate ratio—admixture B

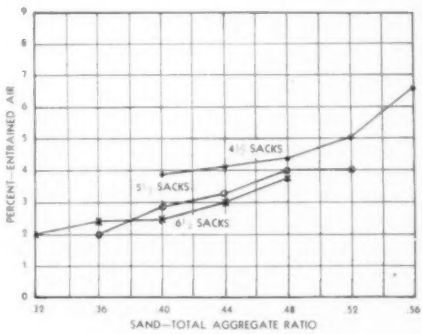

Fig. 8 (right)—Variation in entrained air with sand-total aggregate ratio—admixture C

With the three air-entraining agents the air contents for the 5½ and 6½-sack mixes are generally quite similar for a given sand-total aggregate ratio. In most cases the air contents with the 5½-sack mixes are slightly higher than those secured with the 6½-sack mixes, but this is not universally true. In all cases the 4½-sack mixes give substantially higher air contents for a given sand-total aggregate ratio than do the two richer mixes, which is in confirmation of the previous observations on the effect of cement factor on amount of entrained air.

CONCLUSIONS

1. Of the factors investigated, the two most important with respect to and influence on the amount of air entrained, are cement factor and sand-total aggregate ratio.

2. Variations in slump, within the limits investigated, are of relatively minor importance in determining the amount of air entrained. Air content increases with increasing slump, with the possible exception that with very high slumps air may be lost from the more fluid mixes and the air contents thereby reduced.

3. Entrained air content decreases with increasing cement factor. This effect is most marked in the leaner mixes.

4. The amount of air entrained increases with increasing sand-total aggregate ratio. The only exception to this conclusion appears to be that over a certain maximum rather high air content no further increase in entrained air is produced and, in fact, some decrease may occur.

The experimental approach to indeterminate structures provides additional tools for stress determination and design. Details of the devices used and their applications to typical problems are discussed.

Experimental Aids in Structural Concrete Design*

By R. E. GLOVER†, O. J. OLSEN‡, and CARL ZANGAR§

SYNOPSIS

Statically determinate and indeterminate structures are generally designed by analytical methods. This paper describes several experimental aids which have been used in the design and stress analysis of statically indeterminate structures at the laboratories of the Bureau of Reclamation. These methods are used where analytical procedures cannot conveniently be applied. Described herein are photoelastic methods using both the polariscope and interferometer, the Beggs Deformeter, Stresscoat and electric SR-4 strain gages. Applications are given for each method.

INTRODUCTION

Analytical methods are generally used in the design and determination of stresses in both statically determinate and statically indeterminate structures whether of steel, timber, or reinforced concrete.

There are, however, several established experimental methods which are of considerable aid in the design and stress analysis of statically indeterminate structures. The aids to experimental stress analysis used in the laboratories of the U. S. Bureau of Reclamation in Denver include the photoelastic polariscope and interferometer, the Beggs Deformeter, Stresscoat, and electric SR-4 strain gages. These are used where the analytical procedures are too lengthy or involved or where no practical analytical approach to the problem exists.

PHOTOELASTIC METHOD AND APPARATUS

The photoelastic method of determining stresses in structural models is an optical one employing polarized monochromatic light and trans-

*Received by the Institute Dec. 6, 1948. Scheduled to be presented at the ACI 45th annual convention, New York, New York, Feb. 24, 1949. Title No. 45-24 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 6, Feb. 1949, *Proceedings* V. 45. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than June 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

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parent, stressed models. The theory of ordinary and polarized light may be found in many texts[1,2] and will not be described here. Models employed in this method must acquire the property of double refraction when placed under stress. Sir David Brewster[3] noted as early as 1816 that certain transparent bodies became doubly refractive when stressed. Suitable model materials must be:

a. Elastic; that is, follow Hooke's Law
b. Transparent
c. Easily machined
d. Flat and have a high polish
e. Stress-free when unloaded and easily annealed
f. Highly sensitive optically.
g. Stable for moderate periods of time

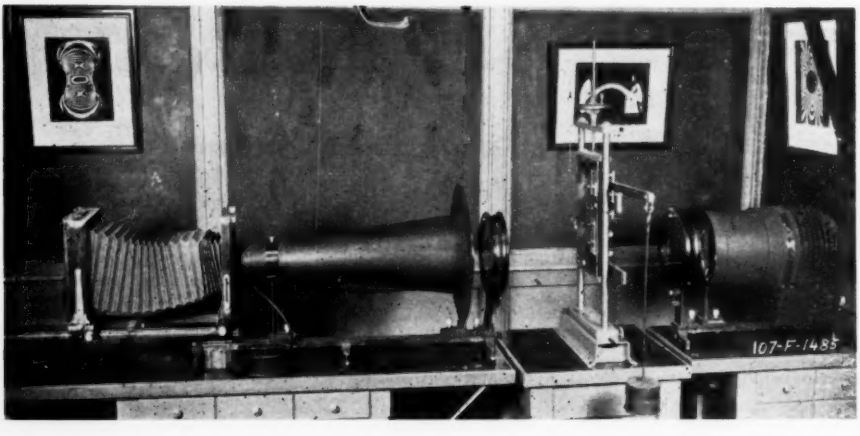
The materials considered most satisfactory for photoelastic models are water-white bakelite, BT61-893, and Allite, CR-39. Other materials used are Celluloid, Marbelette, Catalin, and gelatin.

Two types of photoelastic apparatus are used by the Bureau; the polariscope and the interferometer.

Photoelastic polariscope

The photoelastic polariscope consists basically of a light source and two "crossed" polaroids or Nicol's prisms (Fig. 1). A polaroid or Nicol's prism polarizes light thus permitting it to vibrate in one plane only. If a second polaroid is placed in the light path with its plane of polarization at 90° to the first polaroid no light will be transmitted. These are called crossed polaroids. A stressed, optically sensitive model placed between the two selectively modifies the plane of polarization so that

Fig. 1—Photoelastic polariscope



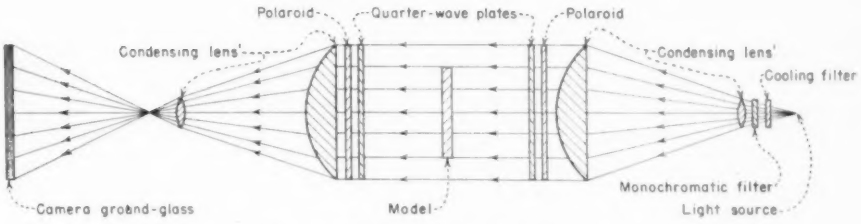


Fig. 2—Optical path of polariscope

light traverses the second Nicol prism to form a familiar stress pattern called isochromatics. Essential parts of the polariscope are shown in Fig. 2. The principal lenses, 10 in. in diameter, permit an effective light field 8 in. in diameter. In using a polariscope it is necessary to insert a quarterwave plate in the light path on each side of the model, as shown in Fig. 2, to obtain circularly polarized light. This is done to eliminate from the stress pattern (isochromatics), certain dark bands called isoclinics. These isoclinics connect all points in the stressed model which have principal stress directions corresponding to the plane of polarization of the light. The isochromatics, or stress pattern, observed in the polariscope, connect points in the model having the same difference of principal stresses. This will be described in more detail below. Monochromatic light is used in conducting experiments since it produces a much sharper stress pattern than white light.

Stress Determination—If a plane-polarized monochromatic light ray is sent normally through a plate stressed in its own plane, the ray will be resolved into two component rays each vibrating in a plane of principal stress. The two ray components will usually have different velocities in the two planes; therefore, while passing through the plate the wave front of one ray will get ahead of the wave front of the other ray a small distance, δ, which is called the relative optical displacement of the two ray components. Now, if the two components are combined after leaving the stressed plate by passing them through a second polaroid so that they will vibrate again in the same plane, interference will take place due to the relative displacement, δ.

Complete interference will occur when $\delta = 0, 1, 2, \ldots N$, wave lengths; thus furnishing a means of measuring the relative displacements. It has been shown theoretically[4] and verified experimentally[5] that the difference in magnitude of the two principal stresses, σ_1 and σ_2, at the line of passage of the ray is in direct proportion to the relative displacement. If a field of plane polarized light is now passed through the stressed model in place of the single ray, bands will form connecting points in the model having the same relative displacement of the light rays or the same difference of principal stresses. These bands or fringes

caused by the interference of the light are known as the isochromatics and form the stress pattern. The equation expressing this optical effect is

$$\sigma_1 - \sigma_2 = NC \quad (1)$$

where

σ_1, represents the maximum principal stress
σ_2, the minimum principal stress
N, the order of interference or fringe order as counted on the isochromatic stress pattern, and
C, is a constant which depends upon the type of model material, the thickness of the plate, and the wave length of the light.

C is easily determined by a simple experiment which will be described later.

From the mathematical theory of elasticity it is known that the difference in principal stresses, $\sigma_1 - \sigma_2$, is equal to twice the maximum shear at a point, or

$$\tau_{(max)} = \frac{\sigma_1 - \sigma_2}{2} \quad (2)$$

and so, isochromatics may be defined as contours of equal maximum shear.

On the boundaries of a model one of the principal stresses is known; it is either zero or equivalent to the applied boundary load. The other principal stress may be computed by use of the stress pattern and Eq. (1) provided the constant C is known.

Calibration—In order to evaluate the model stresses, say in psi, it is necessary to determine the constant C of Eq. (1). A common method is to construct a beam of rectangular cross-section of the model material and subject it to uniform bending. The loaded beam is placed in the polariscope and photographed. By counting, the fringe order N is determined at both the top and bottom surface of the beam and the average recorded. The next step is to determine accurately the dimensions of the beam and the magnitude of the applied load, then the extreme fiber stress is computed by use of the equation

$$S = \sigma_1 = \frac{Mc}{I} \quad (3)$$

where

$S = \sigma_1$, represents the extreme fiber stress
M, the bending moment
I, the moment of inertia, and
c, the distance from the center of the beam to the extreme fiber.

From Eq. (1) with $\sigma_2 = 0$

$$\sigma_1 = NC \text{ or}$$

$$C = \frac{\sigma_1}{N}$$

but from Eq. (3)

$$\sigma_1 = \frac{Mc}{I} \text{ and so}$$

$$C = \frac{Mc}{IN} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Another method of calibration widely used employs a tension specimen cut from the same plate as the model. A direct pull P is applied to the specimen and the fringe order determined at the center cross-section having an area A, then

$$S = \sigma_1 = \frac{P}{A}$$

and in this case

$$C = \frac{P}{AN} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Application—The stress pattern in a conduit section due to a concentrated load at the top is shown in Fig. 3. The boundary stresses obtained by use of the polariscope are shown together with stress patterns obtained from the photoelastic interferometer.

Limitations—Photoelastic stress analyses of two dimensional structures performed in the polariscope give the difference in principal stresses at all points in the model, the boundary principal stresses, and the direction of principal stresses. To obtain the magnitude of the separate principal stresses requires another analytical or experimental procedure for finding the sum of the principal stress $(\sigma_1 + \sigma_2)$ at all points which may be combined with the difference of principal stresses. This procedure is lengthy and much accuracy can be lost in combining results giving the difference of principal stresses with results giving the sum of the principal stresses.

Photoelastic interferometer

The photoelastic interferometer permits the direct determination of the magnitudes and directions of the separate principal stresses at a point in a stressed model. This instrument, which measures the absolute retardation of a beam of light passing through a stressed transparent model, was brought to the attention of the scientific world by Henry Favre[6] in 1929. It is a precise interferometer of the Mach-Zehnder or Jamin type. The Bureau's instrument (Fig. 4) completely

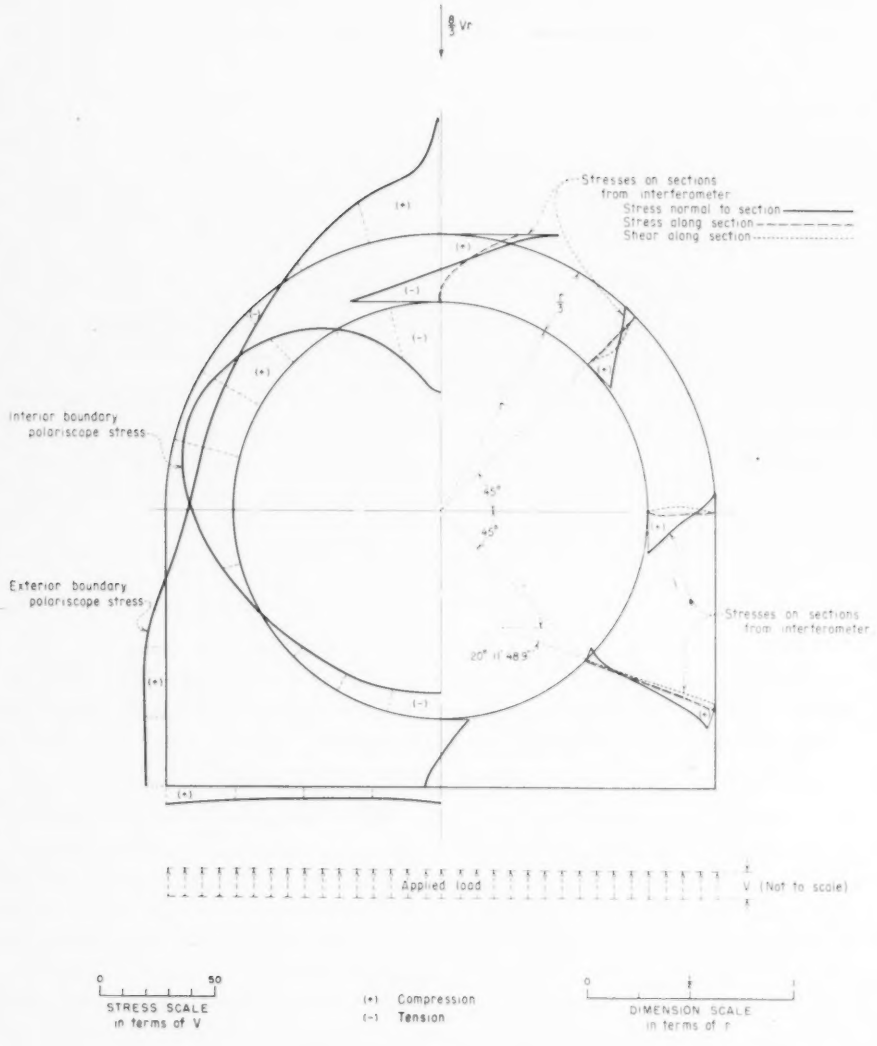


Fig. 3—Stress pattern in conduit section under concentrated load at the top

described in a paper[7] by Brahtz and Soehrens, was patterned after the one built by Favre, but includes several improvements; the main one being a feature which permits finding the direction of the principal stresses. A brief description will be given here since the instrument is not widely known and since it is believed that the Bureau has the only one in operation. It should not be confused with lateral extensometers employing the interferometer principle in their operation.

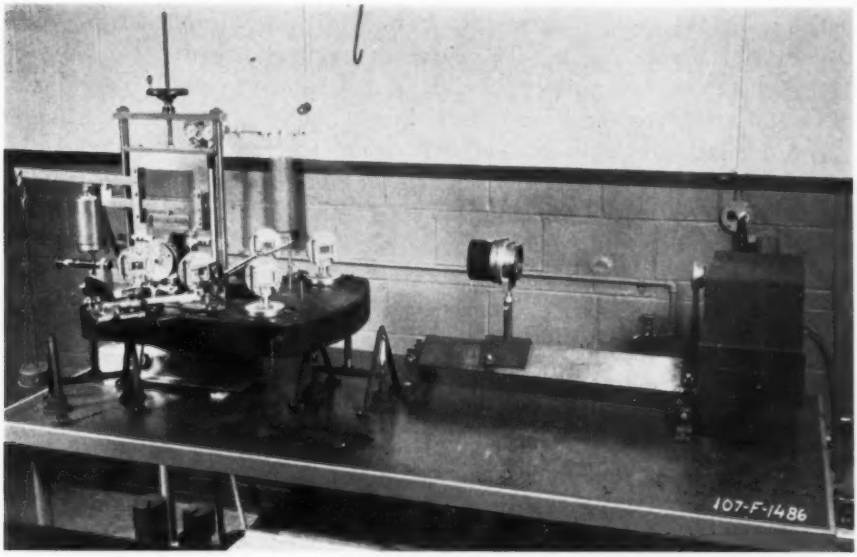
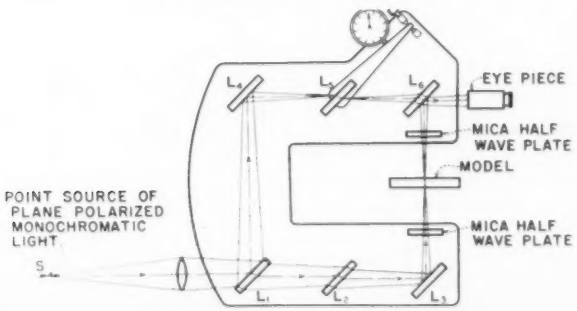


Fig. 4—Photoelastic interferometer

Fig. 5 is a diagrammatic layout of the interferometer. The light source at S is a high-intensity, water-cooled, mercury-vapor lamp. The rays are filtered and passed through an aperture 1/128 of an inch in diameter and through a polaroid giving in effect a monochromatic point source of polarized light. The light next passes through a projection lens so located that it brings the light to a focus in the model. Upon leaving the projection lens the light strikes the optical parallel L_1 which has a thin coating of aluminum on one face. This splits the light into two paths of equal intensity. One path, called the inside path, goes to the full reflecting optical flat L_3, through the stressed model, and thence to L_6. L_6 is an optical parallel like L_1. A second

Fig. 5—Optical path of interferometer



path, called the outside path, goes from L_1 to the full reflecting optical flat L_4 and thence to L_6. The interference phenomenon is developed at L_6 where the two paths converge, and is visible in the eyepiece as a series of alternate light and dark bands called interference fringes. These fringes are entirely different from the familiar isochromatic fringes of the polariscope, particularly in that the interference fringes are visible both before and after loading the model. The interference fringes are merely shifted by a change in stress within the model, and the change in stress is linearly related to the number of interference fringes that cross a fixed point in the field of the eyepiece.

The fringe movement at a point in a stressed model is dependent, in general, upon the plane of vibration of the polarized light passing through the model. It is therefore essential that the model be penetrated by plane-polarized light vibrating in a principal stress direction. This may be accomplished by the use of an auxiliary polarizing disk or analyzer placed in front of the eyepiece at the same time that the light from the outside path is cut off. The two polaroids are crossed so that the light from the inside path is extinguished. However, the vertically polarized light emitting from the light source will not, in general, be vibrating in the plane of the principal stresses and so the second or auxiliary polaroid set to extinguish vertically polarized light will permit the components of light vibrating in the principal stress planes to pass through into the eyepiece. The vertically polarized light may be made to vibrate in a principal stress plane by the use of a half-wave plate placed in front of the model. A second half-wave plate placed behind the model is set to rotate the light back to the vertical plane. The direction of the principal stress is found by rotating the two half-wave plates in unison and noting the angle of rotation necessary to produce extinction while observing the effect in the eyepiece. Having the light vibrating in a principal stress direction, the analyzing unit (auxiliary polarizing disk) is removed from in front of the eyepiece. The interference fringes are again visible and by slowly removing the applied load acting on the model the fringe movement in the eyepiece is noted and recorded.

The laws relating changes in refractive indices to principal stresses have been investigated by such men as Maxwell, Neumann, Favre, and Filon. For plane stresses these relations are:

$$\begin{aligned} \delta_1 &= C_1 \sigma_2 + C_2 \sigma_1 \\ \delta_2 &= C_1 \sigma_1 + C_2 \sigma_2 \end{aligned} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

where

δ represents the change in refractive index,
σ the principal stress,

1 and 2 are subscripts denoting principal stress directions,
C_1 the transverse stress-optical coefficient, and
C_2 is the direct stress-optical coefficient.

Solving equations (6) for the principal stresses, gives

$$\sigma_1 = A\delta_1 + B\delta_2 \quad \ldots\ldots (7)$$

$$\sigma_2 = B\delta_1 + A\delta_2 \quad \ldots\ldots (8)$$

where

$$A = \frac{C_2}{C_2^2 - C_1^2} \text{ and}$$

$$B = \frac{C_1}{C_1^2 - C_2^2}$$

The constants A and B are determined by a simple calibration experiment described later. When these constants are known the principal stresses σ_1 and σ_2 can be determined by Eq. (7) and (8) in terms of the changes in refractive indices which are measured by the number of interference fringes that pass the cross hairs of the eyepiece while the model is undergoing a stress change.

Operation—In actual operation of the interferometer the number of fringes that move past the cross hairs are not counted. Instead the outside path is changed in length by an amount equal to the refractive change of the inside path. This is accomplished by using the clear glass optical parallel L_5 which is mounted on a spindle equipped with a tangent arm and tangent motion. Turning the tangent screw increases the effective length of glass through which the outside light path must travel. The change in length of the outside path may be made equal to the change of the inside path. A micrometer dial bears against an agate insert on the tangent arm, and changes in length of the outside path can be read directly on the micrometer dial. The optical parallel L_2 is necessary only as a compensator for L_5.

Stress measurements are made by first determining the direction of principal stresses and then gradually removing the load acting on the model. As the load is removed the interference fringes move across the field of vision in the eyepiece. The fringe displacement is nullified by turning the tangent screw, and the difference in dial readings before and after loading is a measure of the refractive change δ. The operation is repeated for the other principal stress direction. Magnitudes of the principal stresses are obtained by substituting the difference in dial readings, δ, into Eq. (7) and (8) with the proper values for the constants A and B.

Calibration—The constants A and B are determined by a simple calibration experiment. A tensile specimen is cut from the same plate as the

experimental model, placed in the interferometer and subjected to stress. The principal stress directions are set in succession on the half-wave plates and the dial readings δ_1 and δ_2 obtained in turn by noting the fringe movement between the position of load and no load on the tensile specimen. In a tensile specimen $\sigma_2 = 0$ and, as in the case of the polariscope, $\sigma_1 = \frac{P}{A}$ and so by inserting the values of δ_1, δ_2, and σ_1 into Eq. (7) and (8) the constants A and B may be determined. These constants are functions of the model material, the model thickness and the wave length of the light.

Application—The distribution of stresses across several sections of a conduit loaded with a concentrated force at the top are shown in Fig. 3.

In the Bureau equipment, the loading frame can be used interchangeably in the polariscope and interferometer so that both types of equipment can be brought to bear. This has proved to be a very satisfactory arrangement.

THE BEGGS DEFORMETER

The Beggs deformeter[8] (Fig. 6), devised by the late Prof. George E. Beggs of Princeton University is an apparatus for the mechanical analysis of statically indeterminate structures. Small elastic scale models of structures are deformed by a special set of plugs and gages and the accompanying deformations in the model are accurately read by means of micrometer microscopes. Many model materials have been used such as thin sheets of celluloid, fiber, and cardboard, but sheets of Lucite or Plexiglass about 0.06 in. in thickness have been found to be most satisfactory. The basis of the method comes from a direct application of Maxwell's Law of Reciprocal Displacements which establishes that: given two points on a structure, the ratio of the deflection at the first point to the load causing it, applied at the second point, is equal to the ratio of the deflection at the second point to the load causing it, applied at the first point.

Now consider the indeterminate frame shown in Fig. 7(A). It is fixed at points a and b and loaded at point c with a force P. This structure is equivalent to the structure shown in Fig. 7(B) where the support restraint at point b is replaced by the yet undetermined forces or reactions H, V, and M. H is the horizontal reaction, V the vertical reaction, and M the moment acting at b. The unknowns H, V, and M must be determined so that there is no displacement or change in slope at b. Once these are determined the structure is statically determinate.

To determine the magnitude of the horizontal force, H, suppose momentarily and artificially, that the column base is on frictionless

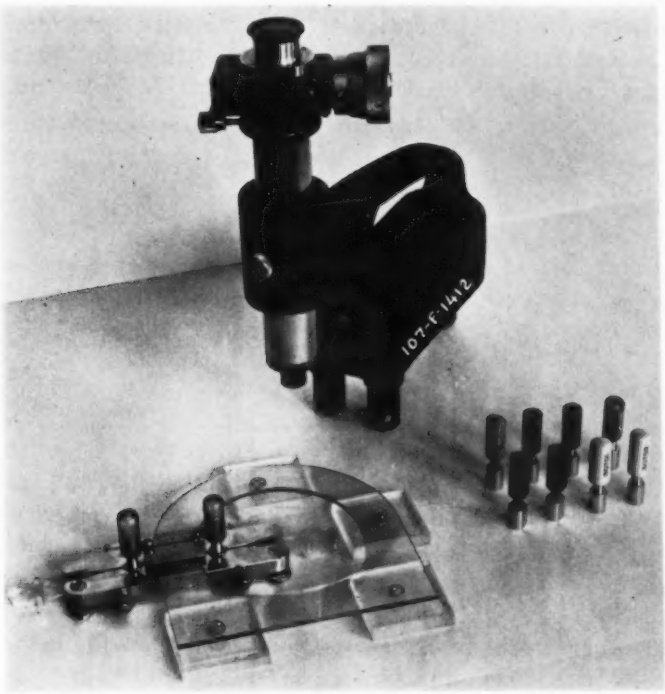


Fig. 6—Beggs deformeter

rollers and therefore free to displace laterally. Under the action of the load P the structure will assume the shape shown at (C) of Fig. 7, and point b will move outward a distance of d inches. Now suppose that the structure is loaded only with the horizontal force H as shown at (D) in Fig. 7. H has a value such that the point b moves inward a distance of d in. and simultaneously the unloaded point e moves upward a distance of e inches. Now if the two systems at (C) and (D) of Fig. 7 are superimposed the result will be the original structure with its original load. Applying Maxwell's Law to the structures and loads shown at (C) and (D) gives the relation

$$\frac{d}{P} = \frac{e}{H} \quad \text{or}$$

$$H = P\frac{e}{d}$$

The reaction or force H is, therefore, equal to the product of the known force P and the ratio of the deflections e and d. The deflections e and d exist simultaneously on the structure shown at (D) and regardless of the magnitude of the force H required to produce the deflection d, the ratio

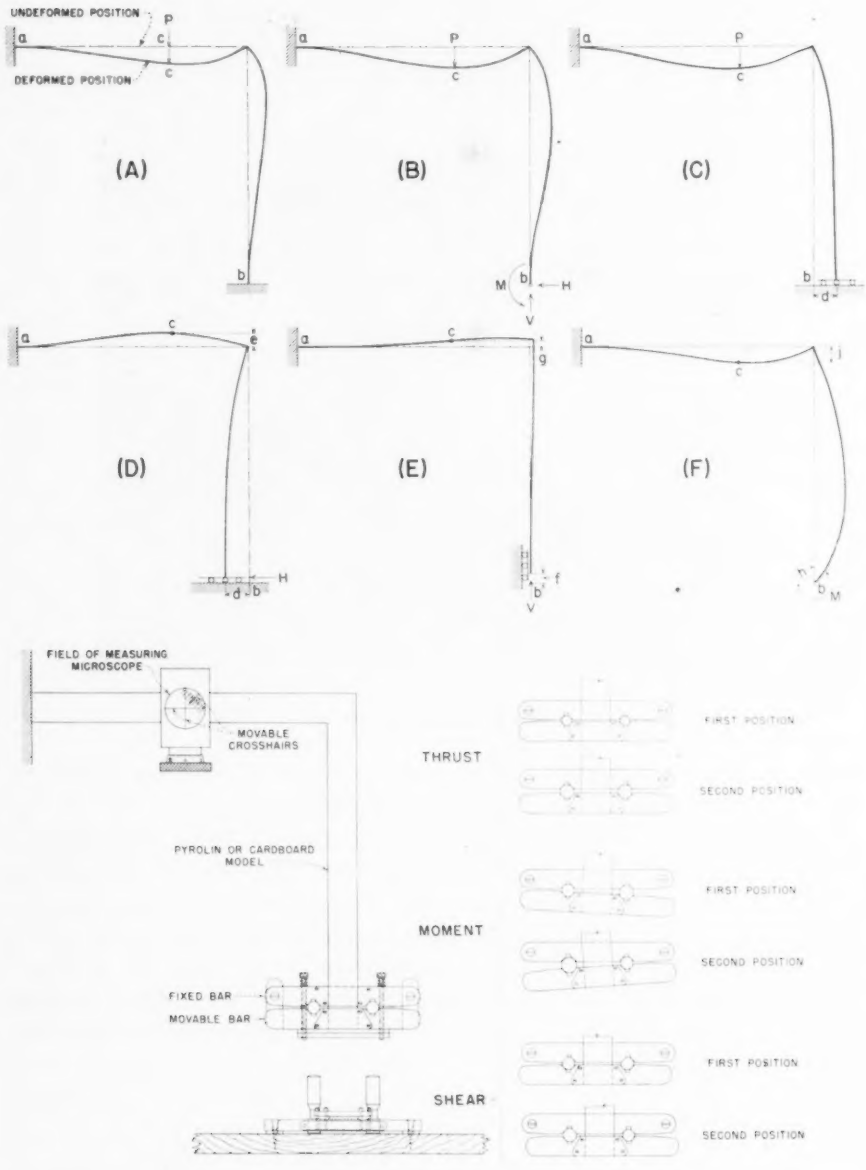


Fig. 7—Deformation patterns of models in Beggs deformeter with details of placing loads

of e to d will remain unchanged. The magnitude of d is therefore quite arbitrary and furthermore the ratio of e to d is independent of the size of the structure. This is the basis of the Beggs deformeter analysis.

By similar reasoning it can be shown that the vertical reaction is expressed by the equation

$$V = P\frac{g}{f}$$

where g is the deflection at the load point caused by the displacement f acting at point b as shown in Fig. 7(E).

The moment M acting at b is given by the relation

$$M = P\frac{j}{h}n$$

where j is the deflection at the load point caused by the angular rotation, h, (measured in radians) as shown in Fig. 7(F), and n is the scale factor. The scale factor is the ratio of prototype dimensions to corresponding model dimensions.

Fig. 7 also shows the method of applying the displacements and rotations to the model by means of the gage attached at the base of the column. The gage is made of two notched steel bars. Accurately ground hard steel plugs are inserted in the notches, and held snugly by two small coil springs. Each of the three types of displacement, corresponding to H, V, and M, requires a separate set of plugs.

The horizontal displacement employed in determining the horizontal force H is obtained with one set of plugs, called shear plugs, which are nearly circular in cross section except for an accurately ground flat side or facet as shown in Fig. 7. The shear plugs are first inserted with the facets facing the upper left, then inserted with the facets facing the upper right. The net effect is a horizontal displacement of the model to the right without vertical or angular movement at the gage.

The vertical displacement employed in determining the vertical force V is obtained by two sets of plugs (thrust plugs) of circular cross section. Two plugs of equal but relatively small diameters are first inserted in the gage, followed by two plugs of relatively large diameters. The net effect is a vertical displacement of the model downwards without horizontal or angular movement at the gage.

For the angular displacement employed in determining the resisting moment M, only one set of plugs (moment plugs) is used. These are of circular cross section but one plug has a diameter slightly smaller than the other. The plugs are inserted first with the small one in the left notch and the large one in the right notch, then reversed in position with the small plug on the right and the large plug on the left. The net effect

is an angular displacement in a counterclockwise direction without vertical or horizontal movement of the gage.

Microscopes equipped with filar eyepieces are used to read the model displacements at points corresponding to the load points of the actual structure. Displacement must be read in the direction of the applied loads. However, instead of actually applying loads to the model, only known displacements at the gage are applied. The difference in microscope readings is a measure of the model displacement induced by the change at the gage from the first position of the plugs to the second position of the plugs.

In the actual operation of the Beggs deformeter, the arithmetic is simplified by the use of calibration factors determined from the plug diameters and the filar eyepiece scales. These factors are such that when multiplied by the microscope readings the product gives directly the ordinate of an influence line or diagram. This influence line gives the magnitude of the moment, thrust, or shear, at the gage position for a traveling 1-lb load.

Application

A Beggs deformeter study was made to determine the moment, thrust, and shear at various points around the cross section of a conduit of the shape shown in Fig. 8. The stress coefficients for this conduit subjected to a uniform vertical load of V psi are given as well as the curves showing the variation in moment, thrust, and shear around the section.

SR-4 STRAIN GAGES

The bonded-wire electric strain gage, commonly known as the SR-4 gage, is useful in experimental stress analysis. This gage, smaller in area than a postage stamp and only slightly thicker, is simply a rectangular grid of fine resistance wire about .001 in. in diameter bonded to a carrier sheet of thin paper by imbedment in a film of elastic cement.

The gage grid with its supporting paper base may be attached by cementing to any suitably-prepared surface on which strain is to be measured. The strain occurring on such a surface, either tension or compression, is transmitted through the paper and cement to the wire, changing its resistance, in part due to the dimensional changes and in part due to stress. The response, which is linear, may be measured by balancing against a known resistance in a Wheatstone bridge circuit, either with a high-sensitivity galvanometer or, more commonly, with a milliameter and vacuum-tube amplifier. It is possible in this way to measure strains of one millionth of an in. per in. This degree of sensitivity, together with the short gage length and small size of SR-4 gages, extends the scope of strain measurement into fields of application here-

NUMERICAL COEFFICIENTS

POINT	$\frac{M}{Vr^2}$	$\frac{T}{Vr}$	$\frac{S}{Vr}$
1	+0.335	+0.033	0
2	+0.286	+0.132	+0.366
3	+0.153	+0.404	+0.633
4	-0.028	+0.773	+0.727
5	-0.207	+1.142	+0.621
6	-0.334	+1.408	+0.343
7	-0.374	+1.500	-0.033
8	-0.349	+1.495	-0.132
9	-0.295	+1.483	-0.230
10	-0.212	+1.464	-0.328
11	+0.020	+0.156	-0.947
12	+0.239	+0.051	-0.633
13	+0.373	-0.012	-0.317
14	+0.417	-0.033	0

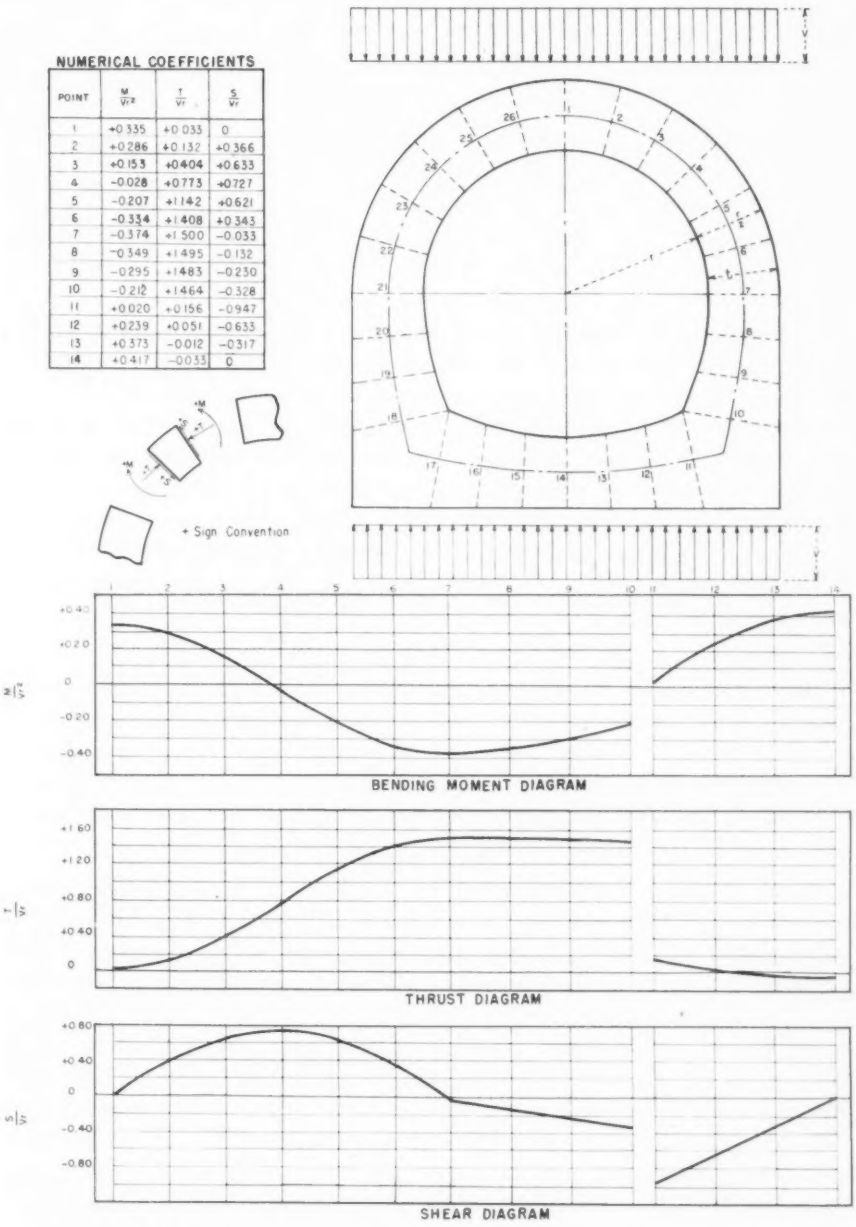


Fig. 8—Beggs deformeter study of conduit to determine moment, thrust and shear

tofore considered impractical, and offers distinct advantages when compared with older types of strain gages. The SR-4 gages are obtainable in gage-lengths ranging from 1/16 in. to 6 in. but the 1/4, 1/2, and 1-in. sizes are the ones most frequently used by the Bureau.

For best results the surface to which SR-4 gages are to be attached must be thoroughly clean, dry, and slightly roughened. Where the gages are to be submerged in water or exposed to a moist atmosphere, waterproofing is essential. Their usefulness on porous materials such as rock and concrete is usually limited to short-time tests because, even though the surface has been thoroughly dried, it is difficult to prevent moisture from rising to the surface beneath a gage and loosening it after a short period of time. In spite of this limitation, however, successful measurements of strain have been made directly on rock and dry concrete in a few instances where dryness could be maintained for the duration of the test. SR-4 gages are adaptable to field testing under conditions of moderate temperature changes, in which case temperature compensation is obtained by use of so-called "dummy" gages. Their use for long-time tests, however, where the elapsed time between application and removal of load exceeds several days, has not proved to be practical according to our experience.

Application

These gages may be used on plastic models of concrete structures to determine the magnitude of surface stresses, but their most important use in structural concrete design in the Bureau has been in the study of steel stress distribution in reinforced concrete structures. Here they may be used to investigate specific current design problems or to increase knowledge of basic design assumptions and methods.

Much progress has been made in the field of testing homogeneous materials, but very little is known about the actual behavior of reinforced concrete after a normal amount of cracking has occurred. Its behavior is known to be different from that of a homogeneous material. On that account the best experimental approach to such a problem is through the use of models constructed of reinforced concrete in which SR-4 gages are attached to the imbedded reinforcing steel at a sufficient number of points to evaluate the effect of cracking on bond strength and stress distribution in the steel right up to ultimate failure. This method of studying reinforced concrete structures is a recent development, and its full potentialities are still far from being realized. A detailed account of the results of concrete beam tests using the SR-4 strain gages may be found in a recent paper by Douglas McHenry and W. T. Walker.[9]

Other applications of SR-4 gages include the measurement of stress distribution in pullout specimens to determine true bond strength be-

tween reinforcing steel and concrete; and high-speed oscillograph recording of stresses during rapid loading to failure. An example of the results of measuring steel stress distribution in concrete pullout specimens by the use of SR-4 gages is illustrated in Fig. 9. Steel stresses computed from strain gage readings are plotted for eight gage points along the imbedded length of a bar at each of five different loads. The gages were attached alternately to opposite sides of a $\frac{3}{4}$-in. square undeformed mild steel bar as indicated at the left of the figure. After waterproofing the gages, the bar was cast in the center of an 8 x 16-in.

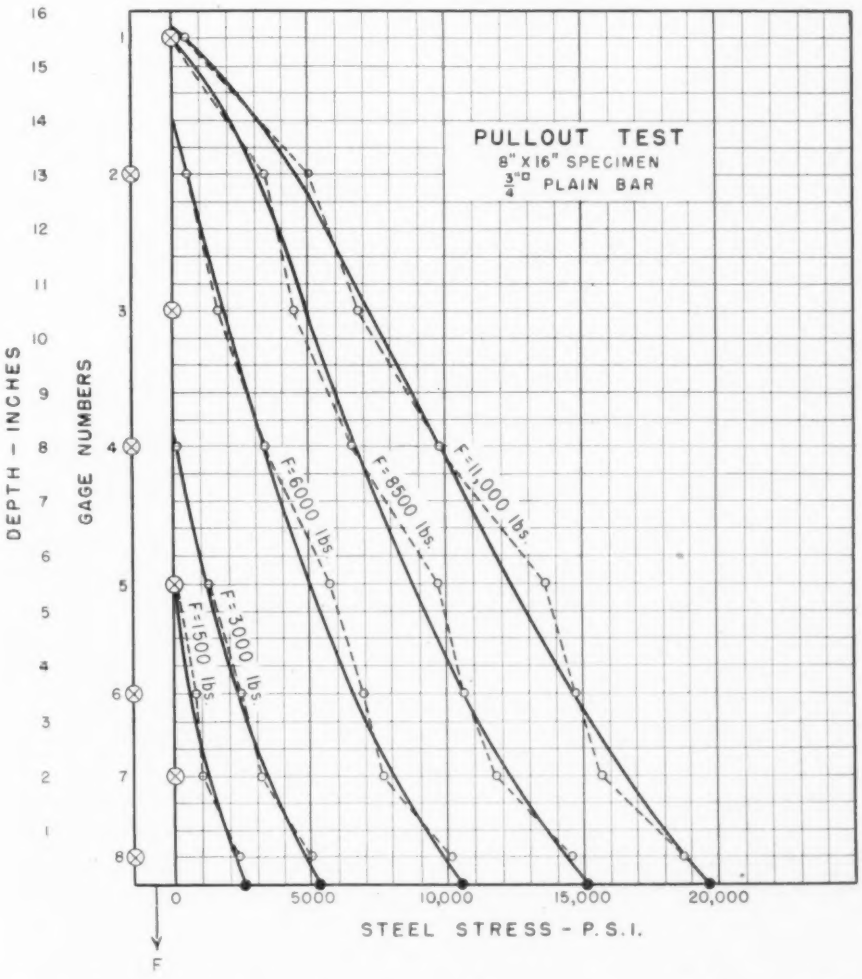


Fig. 9—Steel stress distribution in concrete pull-out specimen determined by use of SR-4 gages

concrete cylinder with one end projecting for gripping. The dotted lines, drawn through the plotted experimental data, show slight deviations from the smooth curves which are believed to be the result of bending in the steel, causing gages on one side to indicate too low a stress while those on the opposite side indicate a higher stress than average. These deviations, no doubt, could have been reduced by using two gages diametrically opposite and averaging the results, but this would require twice the number of gages. The fitted curves indicated by solid lines are identified by the total force applied to the bar, and intersect the horizontal axis at the corresponding values of stress in the steel outside the concrete.

STRESSCOAT

Stresscoat is a commercial name for the brittle lacquer method of stress analysis. It is unexcelled in showing the general stress distribution pattern over the surface of a complex structure, which is the place where failure often originates. It provides a simple and convenient experimental method of determining precisely the direction of principal stresses and a reasonably accurate estimate of stress magnitude. Its particular value in experimental stress analysis lies in the facility with which it can be applied to the surface of 3-dimensional models.

Several techniques for 3-dimensional photoelastic testing have been proposed, but they are somewhat difficult of application, and for this reason stresscoat offers a convenient method for obtaining a knowledge of stress distribution which would be difficult to achieve by other means.

This specially-formulated lacquer, when sprayed to a uniform thickness on a surface to be investigated, will dry overnight to a predetermined degree of brittleness depending on the particular sensitivity of lacquer selected. The coating will then respond to subsequently applied strains by forming crack patterns which are in themselves stress trajectories or isostatic lines. In fact, this is one of the most expeditious means at our disposal for obtaining these isostatic lines. The patterns are basically tension cracks which orient themselves everywhere perpendicular to the direction of the principal tensile stress, but the procedure also can be adapted to investigate compressive stresses. This is done by making practical use of the principle of relaxation and the ability of the lacquer to creep and completely relieve itself of compressive strains in about 3 hours, after which release of compressive load produces tension in the coating. Under progressively increasing loads, these crack patterns reveal the most highly strained areas first, and thus point the way qualitatively toward improving the design in critical sections.

A quick "pilot" test using this method will usually simplify the problem, or at least reduce it to study of a specific location where further

investigation may be concentrated. As applied to the solution of structural concrete design problems, the brittle lacquer method has the limitation that it cannot be used directly on a concrete surface for the reason that concrete, being weaker in tension than the lacquer, would fail before the lacquer could develop crack patterns. This limitation, however, can be surmounted by using scale models constructed of materials more suitable than concrete.

Models are usually made of plastic because its low modulus of elasticity permits the development of large strains with only moderate loads. This also permits investigation of areas of low stress by allowing loads on the model higher than the similitude design load without exceeding the elastic limit of the model material. Plastic models are also easily fabricated. The type of plastic selected should be one that possesses homogeneity, stress-strain proportionality, and low creep properties. Plexiglass and Lucite, both thermo-plastic, and Catalin, a thermo-setting casting resin have been used satisfactorily for structural models.

Prior to application of the brittle lacquer, an aluminum undercoat is sprayed on the surface to give a highly reflective background which is a considerable aid in rendering the cracks more visible. After the lacquer has dried 18 to 24 hours the model is ready for test. Loads are rapidly applied in increments, starting with a value below that which will produce cracking in the lacquer and increasing each increment 25 percent above the preceding total load. At each interval of load the model is examined for crack patterns and then the load is removed for an interval of time equal to twice the time required to load, hold, and unload. This is to eliminate the effect of creep in the lacquer. Stress intensities in the model are determined by drawing contour lines to enclose areas which show cracking for each interval of load. The load-stress relationship is thus established for each of the contour lines on the model. Prototype stresses for maximum load are then determined by simple similitude relationships.

Stresscoat, being rather sensitive to temperature and humidity variations, produces best results when tests are conducted under controlled laboratory conditions, although it is not restricted to such environment. A lacquer having the appropriate sensitivity is selected from a chart coresponding to the anticipated wet and dry bulb temperatures expected to prevail during the test. With no change in the temperature and humidity during the drying and test period, the designated coating should give a sensitivity (*i.e.*, the lowest value at which strain patterns start) between .0007 and .0008 in. per in. strain. Each coating under its designated conditions reacts the same as the others under their designated conditions. Some leeway in the selection is permitted in order to broaden its scope, but unless caution is exercised drying craze

may develop in a lacquer that is too sensitive or brittle; and patterns will close upon release of load if one is selected with insufficient sensitivity. With special techniques, calibration down to .0004 in. per in. without craze is possible, and qualitative pictures of strain direction without calibration have been obtained in areas of strain as low as .0001 in. per in. At the other end of the scale strains up to .0020 in. per in. may be measured by using a colored dye etchant to reveal "hidden" cracks which close after the load has been removed, as in the case of a dynamic test.

The practical limits of strain that can be evaluated by direct calibration, range from .0004 to .0020 in. per in. In terms of stress magnitude, however, the corresponding range is much broader, being determined by the elastic limit and modulus of the model material.

Calibration

Stresscoat must be calibrated for every test either by coating, drying, and testing several calibration strips under the same conditions as the structure being analyzed, or by means of a part of the structure where the stress distribution is known. The accuracy of calibration depends primarily upon the degree to which this condition is fulfilled and secondly upon the matching of the strain pattern on the structure with that on the calibration strip. Accurate matching of strain patterns is feasible only in the narrow range from the least strain which initiates crack formation to approximately .0002 in. per in. strain above that minimum.

The *degree of regularity* of spacing and the length of cracks, rather than the actual dimension of spacing or "crack density," is the best criterion for matching the unknown patterns on the structure with the known patterns on the calibration strip or other calibrating body. Several calibration strips are loaded as cantilever beams by the deflection method throughout the course of a test at the same rate of loading as the model. Match strains are marked on the calibration strips which are then evaluated by placing them alongside a scale supplied with the calibrator.

Application

A typical application of stresscoat to structural concrete design is a 3-dimensional problem involving the determination of torsional stresses in the concrete foundations which support the generators in a powerhouse. In the course of the design of this particular structure, it developed that in the event of a single phase short-circuit in the generator, the foundation would be subjected to a torque load of almost 24 million ft-lb. It was feared that this torque might bring about cracking in the foundation as then designed. To study the effect of such a torque loading a model

study using stresscoat was undertaken, since analytical studies of such a complex structure would at best be only very approximate.

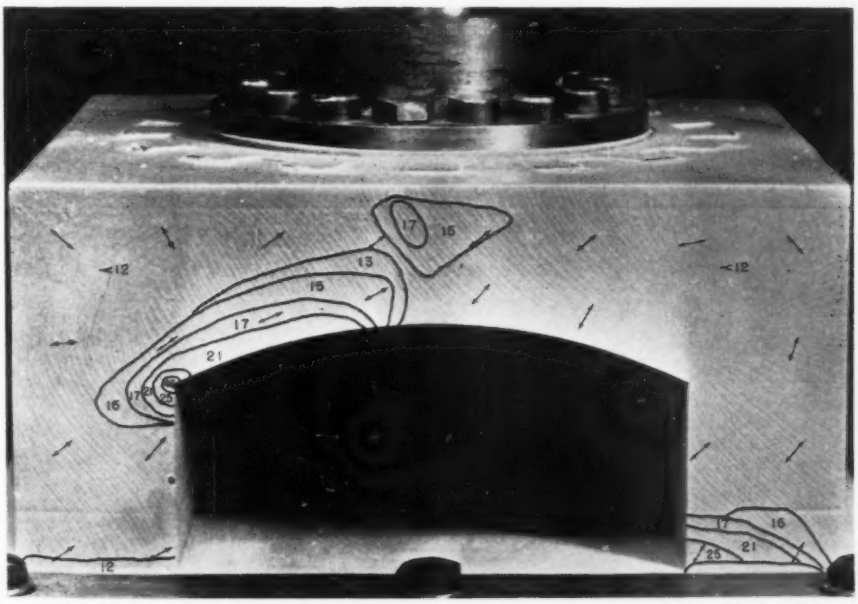
The principal problems in this study were to determine the location of stress concentrations, the magnitude of stress, and the proper direction and location for the reinforcement. A plastic model was used, one face of which is shown in Fig. 10. The directions of the principal tensile stresses (arrows) are everywhere normal to the direction of cracking, and the magnitudes of these tensions are indicated by the numbers within the contours. Tensile stresses as low as 12 psi, which were evaluated by calibration strips, illustrate the order of magnitude attainable by this method. Also, it will be noted that the crack patterns cover practically the entire face of the model including areas where the stress is less than 12 psi. This was achieved by a special chilling technique and is indicative of the qualitative possibilities of stresscoat.

The accuracy of the brittle lacquer method has not been definitely established, but it is believed that under carefully controlled conditions the results may be interpreted in terms of stress within plus or minus 15 percent.

ACKNOWLEDGMENTS

This paper has profited from the helpful suggestions of Douglas McHenry, Howard Kahm, and H. Boyd Phillips.

Fig. 10—Crack patterns in Stresscoat indicating direction and magnitude of principal tensile stress in plastic model



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Air entrainment has been a top preoccupation of the field of concrete engineering since the turn of the decade. Messrs. Blanks and Cordon have sought to make a condensation of the whole matter based on the experiences of the Bureau of Reclamation. Another and different approach to a condensed statement of the uses and effects of entrained air is also published in this issue.

Practices, Experiences, and Tests with Air-Entraining Agents in Making Durable Concrete*

By R. F. BLANKS† and W. A. CORDON‡

SYNOPSIS

The authors discuss the advantages of air entrainment with regard to durability, permeability, workability, reduction in alkali expansion, time saving, reduction in water and cement, temperature rise, strength, abrasion resistance, monolithic lightweight concrete and mass concrete. U. S. Bureau of Reclamation experiences are discussed with regard to pumping, transportation and placing, grading of aggregates, factors affecting the amount of air entrained. Air-entraining agents and air-entraining cements are discussed briefly. There is a brief section on the measurements of entrained air in concrete with a recommended procedure.

USE OF AIR-ENTRAINING AGENTS IN CONCRETE CONSTRUCTION

The use of air-entraining concrete has increased rapidly since it was realized in 1938 that the durability of concrete highways was greatly improved by the presence of entrained air in the concrete. Since then air-entraining concrete has been used in many types of structures, thousands of miles of highway, in over 2 million yards of concrete constructed by the United States Army Engineers, and in many structures built by the Bureau of Reclamation. It is being used in Angostura and Kortes Dams which are being built by the Bureau of Reclamation and in Bull Shoals Dam which is being constructed by the Army Engineers. Hungry Horse Dam, a 3 million cu yd structure now under construction by the Bureau of Reclamation, will contain both a pozzolanic material and an air-entraining agent. Air entrainment is specified for all exposed concrete built by the Bureau of Reclamation in those areas of the country

*Received by the Institute, Oct. 4, 1948. Title No. 45-25 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 6, Feb. 1949, *Proceedings* V. 45. Separate prints are available at 35 cents each. **Discussion** of this paper (copies in **triplicate**) should reach the Institute not later than June 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

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[This paper is based on a lecture by Mr. Blanks before the Harvard Graduate School of Engineering and will be presented as to its highlights only at the Institute's 45th annual convention, New York, Feb. 23, 1949—Ed.]

where freezing occurs. Recent tests have shown that air entrainment improves resistance of some concrete to disruption resulting from wetting and drying, heating and cooling. As this action is encountered in all sections of the country, it is planned that air entrainment will be specified for all concrete placed by the Bureau of Reclamation. Durability is of major importance in concrete construction and throughout the nation there are many structures in which the builder would gladly trade 25 percent of the original compressive strength for a longer period of service.

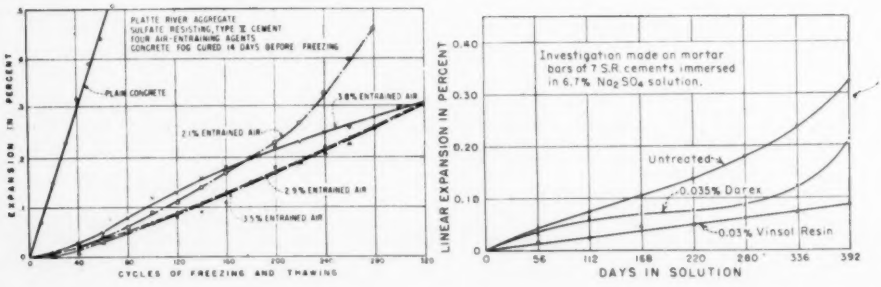
ADVANTAGES

Durability

From field and laboratory experience it can be conclusively stated that air entrainment materially improves the resistance of concrete to weathering and its resistance to freezing and thawing[1] (Fig. 1). It reduces the scaling resulting from the use of calcium chloride for ice removal on highways and improves resistance of concrete to sulfate attack[1] (Fig. 2). Laboratory tests show that durability of concrete containing the recommended amounts of air is on an average about four times that of ordinary concrete containing no air. Even though the strength of the concrete for the same W/C ratio is reduced by as much as 1000 psi because of the entrained air, its durability is still much greater than that of the corresponding concrete containing no air. For ordinary mixes at constant cement content, there is very little if any loss in strength because of the lower W/C ratio resulting from the entrained air. As a word of caution, however, it is known that entrained air is not a "cure-all" and poor-quality concrete, even though improved many times, cannot be made into high-quality concrete through the entrainment of air. (Fig. 3). The optimum air content for any given mix is dependent,

Fig. 1 (left)—Resistance of concrete to freezing and thawing is increased many times by the entrainment of air.

Fig. 2 (right)—Addition of air-entraining agents to concrete increases its resistance to attack by alkalies measured by the expansion of concrete specimens in sodium sulfate solution



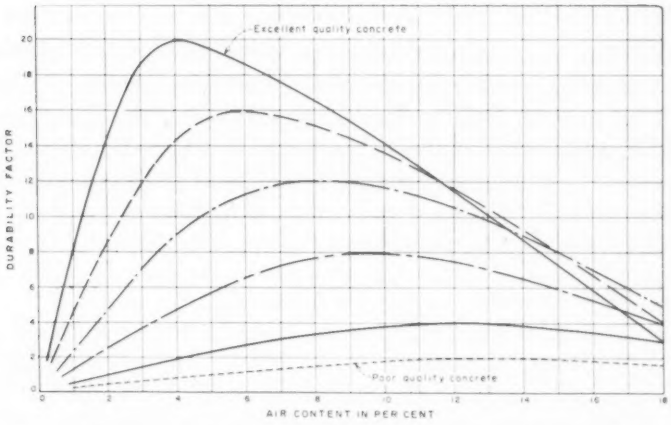


Fig. 3—Optimum air content for durability varies with the quality of the concrete. Durability factor equals the number of cycles of freezing and thawing to failure divided by 100. Differences in quality may be due to difference in W/C ratio or differences in quality of aggregate or cement.

among other things, upon the quality of the concrete and increases as the quality is reduced. The increase in durability for air contents above 6 percent for concrete containing ¾-in. maximum size aggregate does not warrant the consequent reduction in strength for higher air contents.

Various theories have been advanced to explain the improvement in concrete durability due to air entrainment. The most plausible, and probably the most generally accepted explanation, is that the minute, disconnected air bubbles act as a sponge or buffer to provide spaces where the forces which cause concrete disintegration can be dissipated.

Permeability

The permeability of concrete is not materially increased by entrained air[2]. Air-entraining concrete, after it has once dried, is more resistant to the passage of moisture than regular concrete, and it will absorb less water. Small disconnected air voids offer a barrier to the passage of water. Silos for the storage of portland cement showed no caking on the inside when built of air-entraining concrete, whereas those made of regular concrete exhibited the usual amount of caking along the periphery of the silo[3]. This indicates very conclusively that air-entraining concrete is more resistant to the passage of capillary water. Aside from the increased watertightness inherent in air-entraining concrete, the greater degree of uniformity, due to increased workability, in the concrete as placed improves impermeability.

Workability

Entrainment of air greatly improves the workability of concrete, and permits the use of harsher, more poorly graded aggregates, reduces bleeding and, in general, facilitates placing and handling of concrete. The reduced bleeding permits earlier finishing of concrete slabs and usually with less labor. At the same percentage of sand and the same slump, the workability of concrete is increased as the percentage of air in the mix is increased, even though the water in the mix is reduced[3].

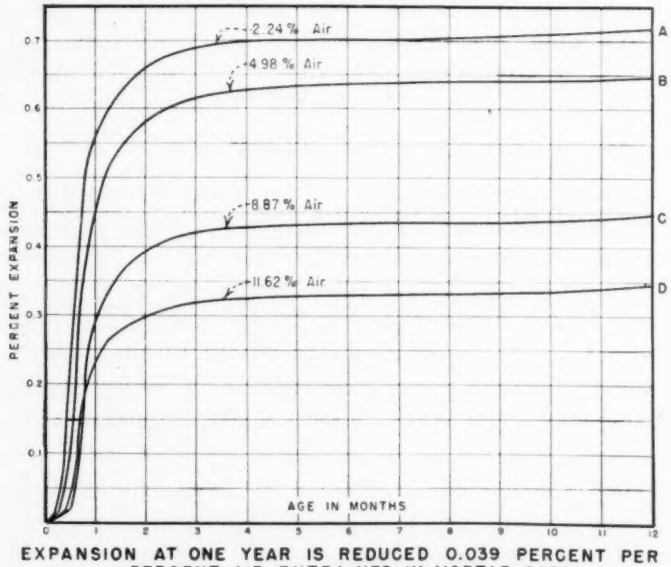
Reduction in alkali expansion

The expansion of concrete due to reaction between the alkalies in cement and certain aggregates is reduced by the entrainment of air[4]. Alkali expansion cannot be eliminated by the use of entrained air, however, and even though the actual expansion may be reduced 50 percent in 1 year as indicated in Fig. 4, the expansion is still excessive and other controls and safeguards must also be employed.

Time saving

Contractors who have been given the option of using regular concrete or air-entraining concrete have usually elected to use the air-entraining concrete and furnish the air-entraining agent. The time saved in placing, handling and finishing more than offsets the cost of a few cents per

Fig. 4—Expansion of mortar bars at one year is reduced 0.039 percent per percent of air entrained.



cu yd of concrete for the air-entraining agent used. Furthermore, there is a saving in material equal, in volume, to the volume of air, which saving cannot be overlooked in economic considerations.

Reduction in water and cement

Each percentage of air entrained permits a reduction in mixing water of from 2 to 4 percent[3]. For a given water-cement ratio, a corresponding reduction in cement content is realized with no loss in slump, with some improvement in workability, with greatly increased durability, but with some reduction in strength. For a given cement content, air-entraining concrete has a correspondingly lower W/C ratio.

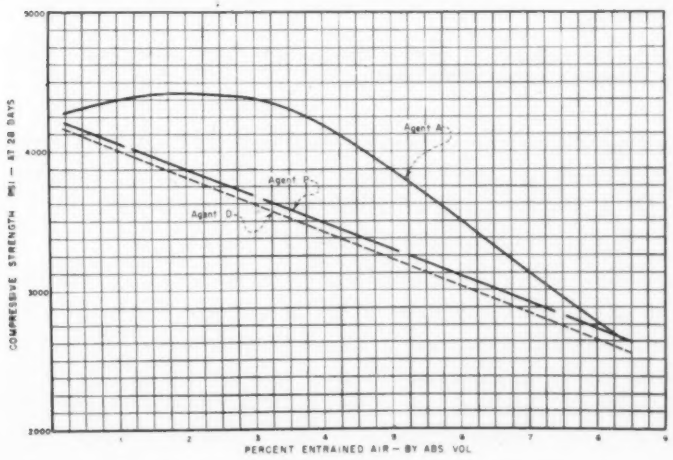
Temperature rise

The reduction in cement for a given W/C ratio results in a lower temperature rise in the concrete. This reduction in temperature rise is of considerable importance in massive structures.

Strength

In a concrete mix containing 1½-in. maximum size aggregate, with a fixed W/C ratio and with the air-entraining agents currently in use, there is a consistent decrease in strength of from 4 to 6 percent for each percent of air entrained[3]. An experimental agent (Agent A) tested in the laboratory shows an increase in strength, however, for lower percentages of air and a more rapid decrease in strength after 4 percent air has been entrained[5] (Fig. 5). For a constant cement content the effect of air entrainment by approved agents on strength varies; for the very lean mixes air entrainment may even result in some increase in strength, the

Fig. 5—With a constant W/C ratio different air-entraining agents produce varying compressive strengths as the percentage of entrained air in the mix is increased.



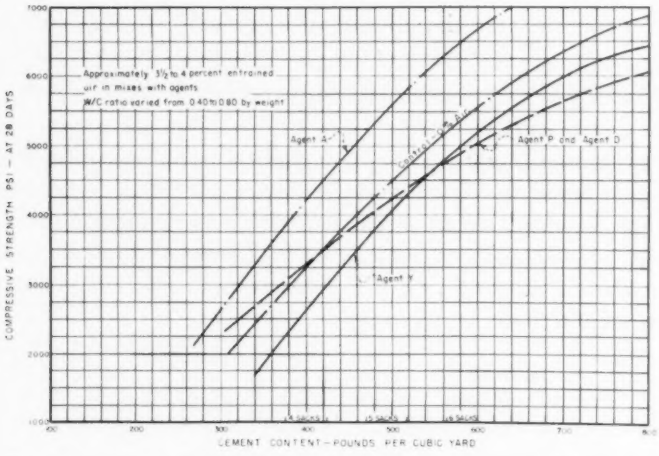
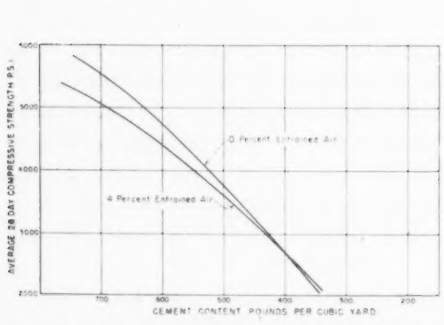

Approximately 3½ to 4 percent entrained air in mixes with agents
W/C ratio varied from 0.40 to 0.80 by weight
Agent A
Agent P and Agent D
Agent Y
COMPRESSIVE STRENGTH, PSI — AT 28 DAYS
CEMENT CONTENT — POUNDS PER CUBIC YARD

Fig. 6—For equal amounts of entrained air and cement different air-entraining agents produce different compressive strengths of concrete.


AVERAGE 28 DAY COMPRESSIVE STRENGTH PSI
0 Percent Entrained Air
4 Percent Entrained Air
CEMENT CONTENT, POUNDS PER CUBIC YARD

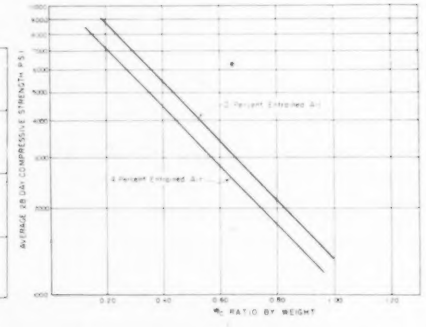


Fig. 7 (left)—With constant cement content, change in strength resulting from entrained air ranges from a slight increase for lean mixes to about 3 percent reduction for each percent of air in rich mixes.

Fig. 8 (right)—At constant air content the strength—W/C ratio curve approximately parallels the curve for concrete without air.

effect varying with richer mixes to as much as 3 percent reduction in strength for each percent of air entrained[6] (Fig. 7). Concrete mixed with Agent A shows a considerable increase in strength over all other concrete, including concrete without air[5] (Fig. 6).

The strength of air-entraining concrete is not proportional to the W/C ratio unless the percentage of air is held constant. If the air content is held constant as shown in Fig. 8, the strength—W/C ratio curve will approximately parallel the curve for plain concrete. There is also a close relationship between the strength of air-entraining concrete and the ratio of the absolute volume of air plus water to cement[7] (Fig. 9).

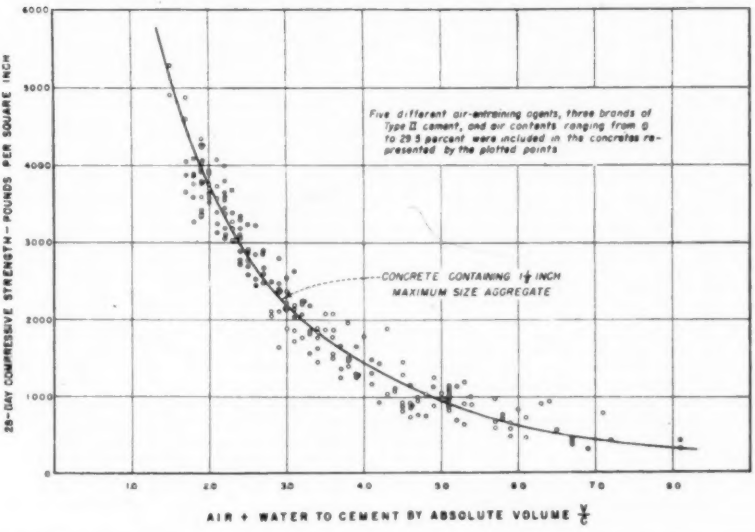


Fig. 9—Strength of air-entraining concrete follows closely the void-cement ratio, voids being equal to the absolute volume of air plus water with cement being measured by absolute volume.

There is reason to believe that the efficiency of air-entraining agents can be materially increased. For example, experimental Agent A indicates that for a given W/C ratio the cement content can be reduced about twice that which is possible with the air-entraining agents currently in use[5]. Expressed in different terms, the experimental agent permits a water reduction of 4 to 6 percent for each percent of air entrained (Fig. 10) which accounts, to a large extent, for the superior strength at constant cement contents. This agent permits a larger reduction in water and cement than the other agents and at a constant W/C there is no reduction in strength for percentages of air up to 4 percent. The experimental agent will probably cost considerably more than the agents now commonly used and it is indicated that greater quantities will be required to give a fixed air content (Fig. 11).

If the strength of concrete is decreased by 10 percent at 7 days' age because of the entrained air, the strength will also be decreased by 10 percent at 28 days' age, and at 365 days' age. In other words, the percentage decrease in strength remains about constant regardless of the age of the concrete[8].

In mass concrete containing 6-in. maximum size aggregate the strength of 6 x 12-in. cylinders, made with concrete wet screened to 1½-in. maximum size, is reduced approximately 6 to 10 percent for each percent of air entrained[8]. This strength reduction, which is approximately 50 percent greater than the reduction for 1½-in. concrete, can be explained

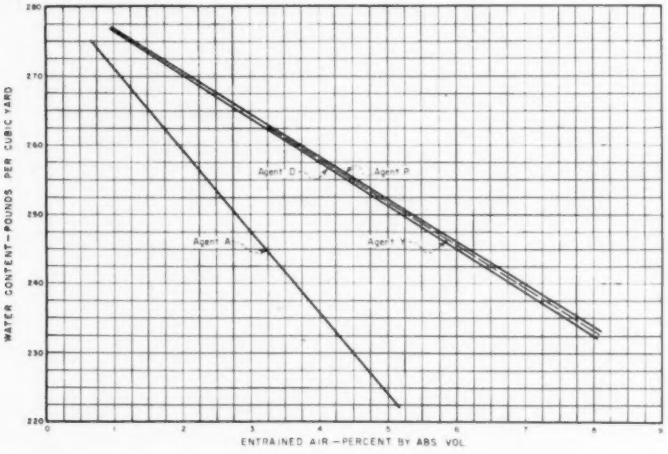


Fig. 10—Different air-entraining agents may give varying water reductions in concrete mixes.

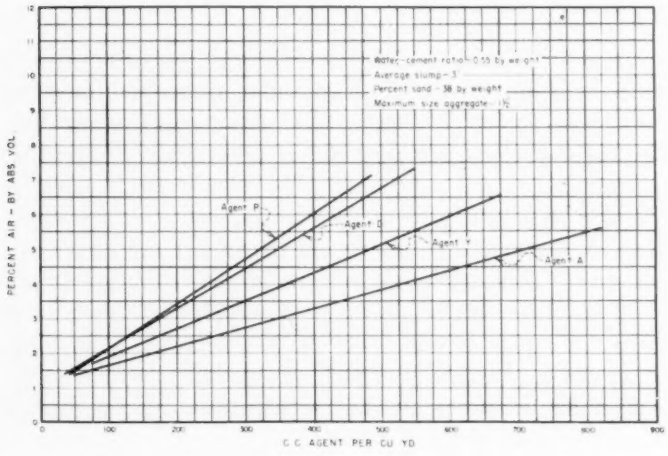


Fig. 11—To entertain a given percentage of air requires different quantities of air-entraining agents.

by the fact that the strength of concrete is proportional to the percent of air in the mortar (water, cement, and all aggregate passing No. 4 screen) sometimes called the air-mortar ratio (Fig. 12). Smaller percentages of air (3 ± 1 percent) in mass concrete, therefore, will produce the desired results for workability and durability and will not reduce the strength unnecessarily.

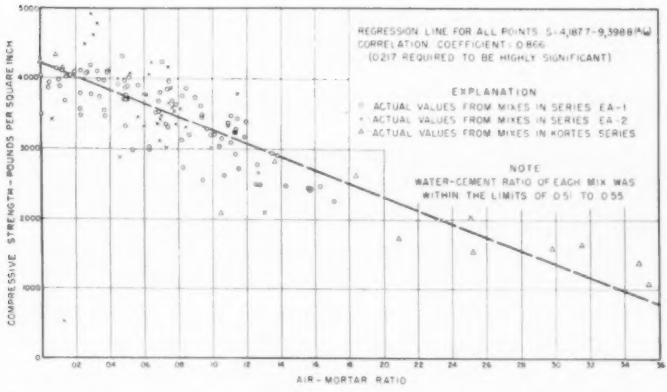

REGRESSION LINE FOR ALL POINTS S=4,877-9,398(A/M)
CORRELATION COEFFICIENT=0.866
(0217 REQUIRED TO BE HIGHLY SIGNIFICANT)
EXPLANATION
ACTUAL VALUES FROM MIXES IN SERIES EA-1
ACTUAL VALUES FROM MIXES IN SERIES EA-2
ACTUAL VALUES FROM MIXES IN KORTES SERIES
NOTE
WATER-CEMENT RATIO OF EACH MIX WAS WITHIN THE LIMITS OF 0.51 TO 0.55
COMPRESSIVE STRENGTH - POUNDS PER SQUARE INCH
AIR-MORTAR RATIO

Fig. 12—Concrete having the same W/C ratio regardless of maximum size aggregate used, is reduced in strength approximately 940 psi for each one-tenth increase in air-mortar ratio.

Abrasion resistance

The abrasion resistance of concrete is influenced more by strength than by any other factor, and where the strength of concrete is reduced because of the entrained air the abrasion resistance is also reduced[9].

Monolithic lightweight concrete

The coarse, porous texture of the majority of lightweight aggregates and the fact that they will float in the cement grout of a plastic concrete mix, makes many lightweight concrete mixes very unworkable and segregation between the grout and aggregate very pronounced. To overcome this condition it has been necessary to add considerable percentages of natural sand or other fines which materially increases the weight of the concrete and decreases its insulating value. The use of air-entraining agents significantly improves the workability, decreases the segregation, decreases the weight, and improves the insulating value of lightweight concrete mixes. Plastic lightweight concrete for use in monolithic construction can be made from many lightweight aggregates with entrained air where plastic concrete with these aggregates was not practical without air entrainment[10].

Mass concrete

Air-entraining concrete has been employed in many structures built in mild climates where its use was not needed for durability but because of the added workability and economy realized from its use. It has already been mentioned that it is being employed in the interior of large concrete dams because of this increased workability and because it permits a reduction in cement content with an attendant reduction in temperature rise. This in turn reduces the tendency to crack as the

concrete subsequently cools and shrinks[8]. It will probably be used in the majority of the concrete dams and major concrete structures constructed in the United States in the future.

Heretofore it has been necessary to include a certain number of fines in mass concrete mixes to insure adequate workability. It has been demonstrated, however, that very workable and placeable mixtures can be made with entrained air entirely replacing the cement. Therefore, the only need for cement in mass concrete for the interior of a large structure is for the production of strength and impermeability. As the stresses are very low in many of the gravity-type dams, it is expected that concrete as lean as 2 sacks of cement per cu yd will be used in some of the future massive structures made with concrete containing 6-in. maximum size aggregate. Laboratory tests with approved agents indicated that the strength of 2-sack mixes containing air will be stronger than those without entrained air (Fig. 6 and 7). Reduction in water content resulting from entrained air is not sufficient to account for this increase in strength, and it may be due to the improved homogenity of the mass and smaller water pockets on the under side of the aggregate particles.

EXPERIENCES WITH AIR-ENTRAINING CONCRETE

Pumping

Experiences in the use of air-entraining concrete when placed by pumpcrete methods vary widely depending upon the individual job conditions and the character of the materials, the concrete mix and other factors. In one case where a tunnel lining was being placed by pumpcrete, it was definitely established that the entrainment of air in the concrete reduced the output of the pump machines, although at the same time there was a noticeable decrease in the pounding of the machines. In this case the concrete was very workable without the entrained air. The only explanation that has been evolved for the reduced output with air-entraining concrete is that the air became sufficiently compressed in the pipeline to cause a backward movement of the concrete when the piston was on the backward stroke.

In another case it was reported that air-entraining concrete actually reduced the rate of feed into the pump and even if all the pipe were removed from the pump, its efficiency would still have been reduced. This condition is probably due to the greater cohesiveness of the air-entraining concrete in comparison with regular concrete.

There have been more reports from actual jobs where pumpcrete methods were being used to the effect that entrained air improved pumping efficiency than there have that it reduced the efficiency. This is

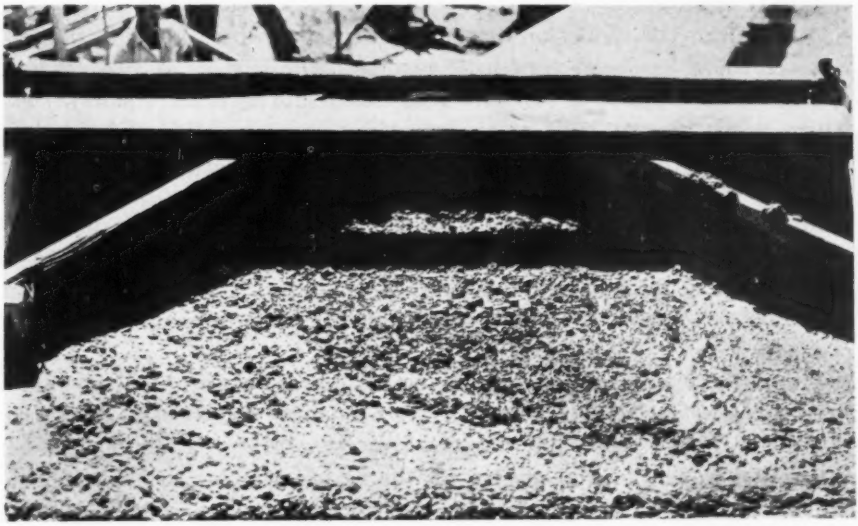


Fig. 13—Air-entraining concrete shows no evidence of segregation after a haul of more than a mile in an open truck

particularly true where the concrete is naturally harsh or low in workability. Experience seems to indicate that for concrete placed by pumpcrete methods, the sand content for air-entraining concrete should be reduced only about half the reduction normally used for ordinary placing methods; that is, 0.5 percent instead of 1 percent for each percent of air entrained.

Transportation and placing

Air-entraining concrete can be transported and placed with less segregation than regular concrete. In fact, it has been hauled in nonagitating trucks for distances up to 15 miles without serious separation of the ingredients (Fig. 13). The length of satisfactory haul, of course, is dependent upon the condition of the road, the temperature and the mix being employed on the job.

Placing procedures and mechanical vibration may reduce the amount of entrained air by more than one-third of that contained when it was dumped from the mixer. However, the vibration, or spading, needed for proper compaction should not be limited for this reason. Instead, measurements of the air content should be based on concrete receiving typical job treatment; the amount of air-entraining agent should be adjusted to provide the recommended amounts of air after the concrete is in final position.

Grading of aggregates

Apart from the reduction of sand percentages where air entrainment is used, there has been very little change to date in aggregate grading specifications because of its use. Experience and tests have shown that where a constant amount of air-entraining agent is used, such as that contained in cements supplied with the agent interground into the product, excessive fines in the sand will reduce the amount of air entrained in the mix, and for that reason some engineers have preferred using coarser sand for air-entraining concrete. Even with fine sands meeting the usual grading requirements, however, it is usually possible to obtain the desired amount of air in the mix by increasing the amount of agent.

Workable air-entraining concrete can be obtained with coarser sands than those normally permitted. Tests conducted in laboratories of the Bureau of Reclamation have shown that any one size fraction can be eliminated from the sand or that all the material below the No. 16 size can be removed and a placeable mix still obtained for concrete containing about 5 percent air. In general, poorer graded aggregates can be used for air-entraining concrete, but until more experience is gained in its use, it is not recommended that grading specifications be broadened for air-entraining concrete.

Amount of air entrainment

As already discussed, the majority of air entrained in concrete is contained in the mortar. The percentage of mortar in the concrete decreases as the maximum size of the aggregate is increased, and it has been found that it requires a lower percentage of air for a mix containing 6-in. maximum aggregate than for one containing ¾-in. maximum aggregate[8]. Tests have revealed that when the air content of a concrete containing 1½-in. maximum aggregate drops below 2 percent, the resistance to freezing and thawing decreases appreciably. There is no advantage, however, in increasing the air content above about 6 percent because, as has already been pointed out, the strength is usually decreased with increased air content, and usually the durability of concrete is not materially increased for air contents above 6 percent (Fig. 3). Furthermore, even though the water is reduced with the entrainment of air, the drying shrinkage is increased for percentages of air above 6 percent. Surface voids which collect against the form become more pronounced as the percentage of air in the concrete is increased. However, there is no appreciable increase in these voids for air percentages up to 5 percent.

Factors affecting the amount of air entrained

There are many factors which affect the amount of air entrained in concrete. With some agents, the amount of air entrained in the mix is

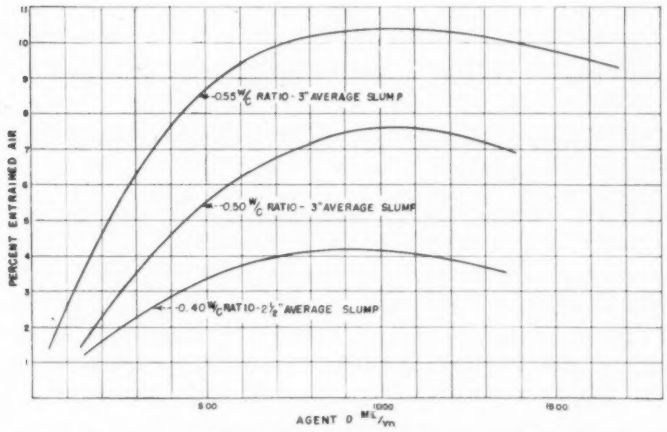


Fig. 14—Air-entraining properties of commercial agent D

proportional to the amount employed, as much as 30 percent of air has been entrained in laboratory concretes. With other agents it is not possible to entrain more than 12 percent air in a mix, and in the case of very rich mixes not more than 5 percent (Fig. 14). After the maximum is reached, additional amounts of agent will decrease the amount of entrained air[11].

Slump, grading of the aggregate, particle shape of the aggregate, temperature of the concrete, mixing time, and richness of mix all have an important influence on the amount of air entrained for a given amount of agent. For different agents, different amounts are required to entrain a given percentage of air[5]. However, after the agents have been properly diluted the amount required to produce about 4 percent air will generally range from ½ to 1 pint per cu yd of concrete. For a given amount of agent, the amount of air entrained in the mix is increased as the slump is increased. Excessive amounts of fines in the sand decreases the amount of air in a mix in the same way as does increasing the cement content or adding pozzalans or other fines to the mix. Less air is entrained at high temperatures than at low temperatures, and it can be expected that the air content of mixes made during the summer will be less than those made during the fall and spring for a given amount of agent. Long periods of mixing will decrease the amount of air in a mix. Where poorly neutralized Vinsol resin or air-entraining cements containing such an agent are used, however, the opposite effect may be noted.

AIR-ENTRAINING AGENTS

Type

Generally speaking, all air-entraining agents fall into one or more of the following classes:

(a) Natural wood resins and their soaps.

(b) Animal or vegetable fats or oils, their fatty acids and their soaps.

(c) Alkali salts of sulphonated or sulfated organic compounds such as a synthetic detergent.

From this general classification it may be seen that there are innumerable materials which will entrain air. Such materials vary widely in their effects on concrete for reasons not too clearly understood. Some differences might be explained by the solubility or insolubility of the calcium soaps or other compounds formed. At the present time there are two air-entraining agents approved for use when interground with cement in accordance with A.S.T.M. specifications for cement. The trade names of these two materials are Darex, a material falling in class (c), and Vinsol resin, a material falling under class (a). These two materials are also quite generally used at the present time as an admixture added at the concrete mixer.

Darex

Darex is a triethanolamine salt of a sulphonated hydrocarbon, which is marketed as a liquid for addition at the mixer. It is usually batched as a gage water addition having a density of 8.7 lb per gal.

Vinsol resin

Vinsol resin consists substantially of the petroleum-hydrocarbon insoluble fraction of a coal-tar hydrocarbon extract of pinewood. It is insoluble in water so that it must be mixed with caustic (sodium hydroxide) to give a water-soluble product. It is marketed as a dry powder, which must then be mixed with sodium hydroxide in order to prepare a solution for use at the job. Several distributors now market prepared solutions containing 1 lb of Vinsol resin per gal. of 2 percent solution of sodium hydroxide. This liquid is then used as a gage water addition at the mixer.

Other agents

In recent years a number of agents proposed for use as concrete admixtures to entrain air have appeared on the market. No doubt many will prove satisfactory after tests and experience with them.

There is reason to believe that much more efficient air-entraining agents will be developed through proper combination of air entraining, wetting, accelerating, and other chemicals. By more efficient air-entraining agents is meant materials which will not reduce strength to an objectionable degree and which will reduce water requirements more effectively.

Corrosion of containers

Either Vinsol resin or Darex can be satisfactorily stored, transported, and handled in steel drums. There have been some complaints from field jobs that the materials corroded the steel containers. This corrosion was due to the use of dissimilar metals in the drum plugs which caused galvanic action and thereby induced corrosion. Corrosion troubles were easily cured when this information became known.

Evaluation of air-entraining agents

Recently A.S.T.M. Committee, C-9, published a proposed method for evaluating air-entraining agents. The proposed method is based on comparative relationships to untreated concrete or to approved materials. Evaluation is on the basis of standardized laboratory tests and is not intended to simulate actual job conditions. Briefly, the tests are as follows:

Bleeding—Not more than 65 percent of the bleeding of similar tests on concrete without admixture when tested in accordance with proposed A.S.T.M. methods.

Compressive strength—Not less than 90 percent of a similar concrete without admixture at ages of 2, 7, 28, 90 and 365 days.

Flexural strength—The same as compressive strength.

Resistance to freezing and thawing—Not less than 80 percent of that given by approved agents at 200 cycles.

Bond to steel at 28 days—Not less than 90 percent of similar concrete without admixture in a pullout test.

Length change—Not more than 110 percent of similar concrete without admixture at specified age.

AIR-ENTRAINING CEMENTS

In some cases the air-entraining agent is interground with the cement at the time of manufacture, and is added at the same time as the gypsum during the grinding process. Air-entraining concrete is produced with this type of cement without further addition of materials or agents. The Bureau of Reclamation has not used air-entraining cement primarily because adequate control is not possible. When an air-entraining cement is used, the mixing water must first dissolve the agent before entrainment is accomplished and it may be difficult to completely dissolve or disburse dry air-entraining agents which are not readily dissolved within the short mixing period. With this type of cement, ordinarily a longer mixing time produces more entrained air.

An example of difficulty in using air-entraining cement was experienced with transit mixers in the early days of air-entraining concrete. Cement which contained Vinsol resin which was not completely neutralized sup-

posedly would produce approximately 4 percent air, but when this cement was included in the concrete mix, the excessive length of mixing time during the hauling of the concrete in the transit mixer produced as much as 14 percent entrained air[3]. Vinsol resin which is now used in the manufacture of air-entraining cement, is preneutralized before addition. Where the dissolved air-entraining agent is added with the mixing water, it becomes completely disbursed immediately upon entering the mixer and the full amount of air is immediately entrained in the concrete mix. The use of air-entraining cements is not recommended where facilities are available for accurate control of air entrainment at the mixer. However, if control at the mixer is not practicable, air-entraining cements should be used.

MEASUREMENT OF ENTRAINED AIR IN CONCRETE

Methods

The entrained air in a concrete mix is measured by one of three general methods: Gravimetric, pressure or displacement. The gravimetric method, A.S.T.M. Designation C-138-44, involves a comparison between the computed unit weight, assuming no air, and the measured unit weight of the concrete. The pressure method makes use of the pressure-volume relationship of Boyle's law, and a number of pieces of apparatus have been developed and placed on the market which are commonly known as air meters. The displacement method involves the displacing of entrained air with water, or some liquid, in a measured volume of concrete. Entrained air can be measured by any of the three methods if proper precautions and care are exercised[12].

Job measurement of the air content of concrete may vary widely from the actual value due to variations in samples, inaccuracies in test methods, and careless performance of the tests[12]. It is indicated that the displacement method is less reliable than the pressure or gravimetric methods because extreme care must be exercised in removing all of the entrained air to make the measurement sufficiently accurate by the displacement method.

In the control of concrete on Bureau of Reclamation projects, it is required that standard unit weight tests, A.S.T.M. Designation C-138, be performed so that an accurate determination of the water content and cement content of the concrete mix can be made. By simple calculations or use of a nomographic chart (Fig. 15) the air content can also be determined from the unit weight without further testing. However, unless a careful check is maintained on the specific gravity of the aggregates and the moisture contents of the various materials entering the mix, erroneous results can easily be obtained by this method. Although air meters used in pressure methods will probably produce the

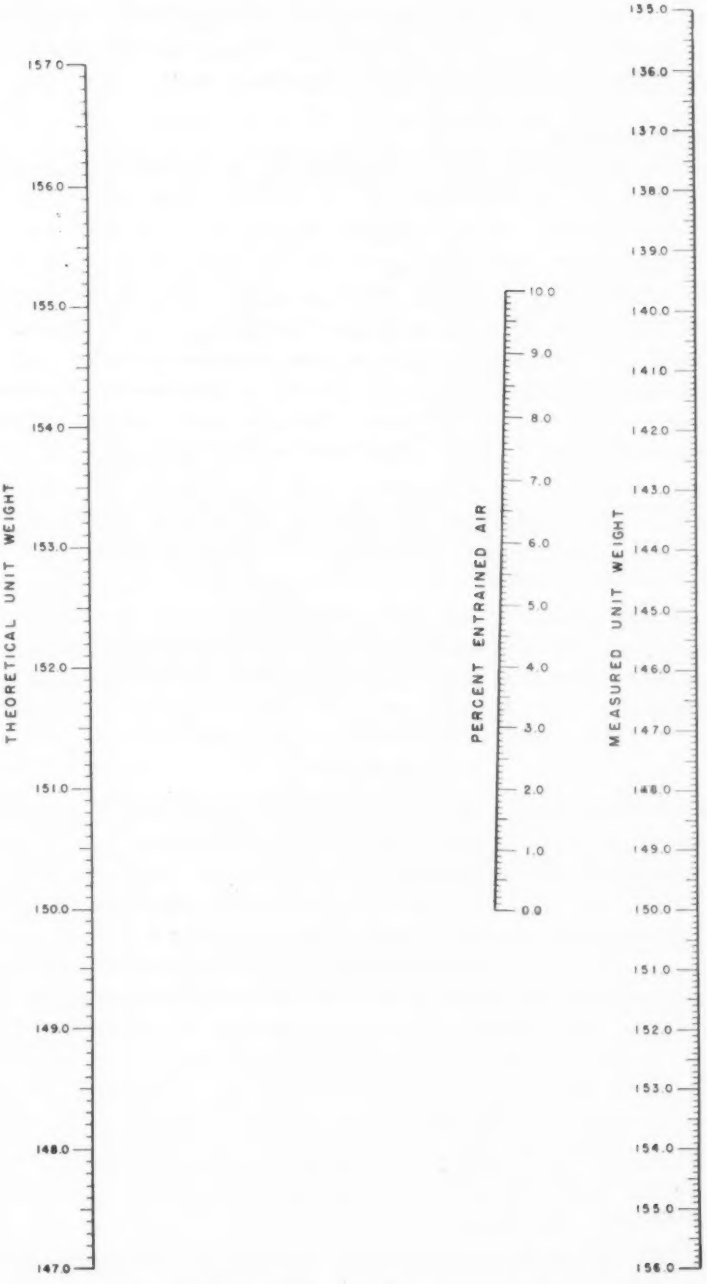


Fig. 15—Chart for the determination of air content of fresh concrete

most consistent results in determining the air content, this is the only property of the mix that is measured with this apparatus unless a sufficiently large meter is used in which the unit weight test can be accurately determined before the air measurement is made.

Reommended procedure

Valuable information can be obtained by combining gravimetric and pressure methods to determine the air content and comparing results of the two methods. If, for example, the air meter is sufficiently large so that an accurate unit weight test can be made, it is simple to make the unit weight test, assemble the apparatus, make an air determination, and record the air content by both methods. An apparatus which has proved to be suitable for this purpose, even for concrete containing 6-in. aggregate, has been built in the Bureau of Reclamation laboratories. Any discrepancy between the two values for air content indicates an error in mix design, physical properties tests of the aggregate or batch weights. In addition to determining and checking the air content and cement and water contents, this method provides a check on the accuracy of mix control. This procedure is therefore recommended for the measurement of entrained air in concrete.

The percent entrained air in a concrete mix can be read directly in Fig. 15 by placing a straightedge on the theoretical unit weight, which remains constant for a constant uniform mix, and the measured unit weight.

SUMMARY

Experiences to date with air entrainment have shown that the advantages in its use far outweigh the disadvantages. Entrained air greatly improves the durability of the concrete, improves the workability of the mix, permits a savings in materials and time, reduces the passage of capillary water, reduces the temperature rise in concrete, increases the strength of lean concrete, permits the use of poorer graded aggregate, reduces the bleeding of concrete, reduces segregation during transportation, and reduces expansion resulting from alkali-aggregate reaction. Objectionable features of air entrainment are: slightly increased drying shrinkage, reduced strength for richer mixes, reduced abrasion resistances when strength is reduced because of air, possible reduced bond to old concrete and to reinforcement, and greater difficulty in controlling mix in the field. Of these the reduction in strength is the only one of any significant importance.

Additional information is needed on the effect of entrained air on the bond of concrete, on the permeability of lean mixes containing air, and on changes which can be safely made in aggregate specifications because

of entrained air. For instance, should we permit the use of aggregates of poor physical quality and extend our grading limits?

Although the improvements resulting from the use of air-entraining agents have been great, experimental agents now under tests indicate that even further improvement in the properties of concrete may be realized. These improvements, combined with advancements in construction procedures, will extend the life of concrete manyfold.

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An impressive body of opinion is to the effect that workmanship in many applications of concrete is a most important factor—in nothing perhaps more important than in floors. Certainly, knowledge adequate to make good floors is available but not generally applied on the job.

Concrete Floor Finishing*

By GERALD MILSOM†

SYNOPSIS

A concise discussion of the requirements for durable concrete floors including careful selection and control of materials and attention to weather conditions. The importance of proper equipment, working conditions and curing of the finished concrete are emphasized.

GENERAL

Some of the requirements for good concrete floor surfaces are:

(1) Skilled supervision and workmanship.
(2) Careful selection of aggregates.
(3) Rigid control of water-cement ratio.
(4) Satisfactory working conditions.

Note that "Skilled supervision and workmanship" is placed first on the list. So often the high standards which should be obtained from excellent design and specifications are lost by inadequate or inexperienced supervision. Each concrete floor presents its own problems requiring skill and experience to solve.

The practice of pouring concrete floors during the morning period only is considered to be a good one. It not only enables the contractor to use his laborers in the afternoon to prepare for the next day's pour but it reduces fatigue, tends to eliminate night work for the finishers, enables the engineer to supervise the finishing to a greater extent, and it often reduces costs by eliminating overtime work.

MATERIALS

The importance of the careful selection of sand and stone, both in quality and size grading, cannot be over emphasized. In fact, it is re-

*Received by the Institute January 13, 1948. Title No. 45-26 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 6, Feb. 1949, *Proceedings* V. 45. Separate prints are available at 35 cents each. **Discussion** of this paper (copies in **triplicate**) should reach the Institute not later than June 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

†Member American Concrete Institute, President, Metallicrete Floor Co., Ltd., Montreal 3, Que., Canada.

commended that all sand and stone be analyzed by a recognized testing laboratory, and the approval of competent authority obtained prior to their use. A slump test or testing for compressive strength does not necessarily determine that the aggregates being used are the most desirable for concrete floors. A careful survey into the importing of aggregates, if the local source is below standard, will often determine that such a procedure is not prohibitive in cost.

The use of metallic hardeners has become a standard practice in many cases, and they have a definite place in producing more durable concrete floor surfaces, provided they are applied correctly. Care should be taken to insure that the additional cost of metallic hardeners is not absorbed by reduction in strength or quality of the concrete to be used for the floor.

Admixtures are sometimes used as lubricants to make placing easier, but they are not "cure alls." Good practice in laying concrete floors should be adhered to regardless of the use of admixtures. Air entraining agents such as "Darex" have been used satisfactorily in concrete for floors, making it possible to place much drier mixes, which in itself has considerable merit. However, our experience with air entraining agents has not been over a sufficiently long period to comment on any change in the durability of the concrete in which they have been used.

WATER-CEMENT RATIO

The practice of adding additional water to the concrete so that it can be more easily handled is general but undesirable. Changing the water-cement ratio should only be done after careful consideration and only with the consent of the inspector. Rigid control of the water-cement ratio determines ultimate strength, reduces fines at the surface and produces a more even texture throughout. Drivers of transit mixers should not be permitted to add tempering water en route, unless they have been given definite written instructions. An approved water meter should be used on all concrete mixers to insure control of the consistency of each batch. Small changes in water control produce marked difference in the workability of the concrete, and a surprisingly small increase in the water will make the concrete too wet, producing spotty setting and uneven textures in the floor surface.

TEMPERATURE

It is recognized that the ideal temperature for pouring concrete is from 70 F to 80 F. Bearing this in mind it is hard to understand why so many concrete floors, designed to withstand wear and abrasive action, are

poured at temperatures of from 32 F to 110 F when such action is not necessary. Careful planning and, at times, a slight increase in cost for more protection, will often overcome this unsatisfactory situation. Information is obtainable showing the strengths of concrete poured and cured at different temperatures, and this should influence the contractor against pouring concrete floors in extreme temperatures.

The heating of aggregates and water earlier in the fall and later in the spring is desirable in colder climates. It is a recognized fact that frozen materials should not be used, neither can good floors be produced from concrete which is so cold that its setting time is considerably delayed.

EQUIPMENT AND WORKING CONDITIONS

Working conditions have a definite effect on the finished product; workmen cannot be expected to produce the best results if the conditions are unsatisfactory. It is emphasized that the toll taken out of floors laid under poor conditions is paid for time and again by higher maintenance costs.

Adequate overhead lighting is required during the laying and finishing of concrete floors. It is impossible for the finishers to produce a superior surface if they cannot clearly see what they are doing. Lighting which produces shadows should be avoided.

The use of mechanical equipment such as vibratory screeds, power floats and finishing machines make possible the screeding and finishing of drier, harsher concrete thereby producing superior floor surfaces. Their use should be obligatory for large floor areas.

CURING

Curing is a very important requirement in producing satisfactory floors, and it is one which is often neglected. For this reason, in general, a clear sealing compound is recommended. The material is easily applied, its effects are lasting because no further attention to the curing is required, and it is economical. Moist curing, the use of non-staining sand, or waterproof curing paper are ideal provided strict supervision is given to insure that the curing is carried out thoroughly during the required period.

Very often finished concrete floors are used before they have thoroughly set. This is often done without protecting the floor, and the damage caused thereby is considerable. This is one reason why a topping, laid after the other trades have completed their work, is desirable. Cooperation between the owners, the general contractors and the flooring sub-contractor will reduce this condition to a minimum, and eventually save the owner from needless expensive repairs.

SUMMARY

Concrete floors with long life and little or no maintenance are being constructed daily. They are economical, satisfactory, and stand up under heavy traffic. With the cooperation of all concerned to insure that the best construction practices are followed, some of the existing problems can be eliminated.

Title No. 45-27

Professor Shank reevaluates data on plastic flow of concrete and relates his findings to the consideration of factor of safety.

Plastic Flow of Concrete at High Overload*

By J. R. SHANK†

SYNOPSIS

New data on the plastic flow of concrete prompted a reconsideration of the assumption that plastic flow is directly proportional to the unit stress applied. Tests are described in which deformations of 6 x 12-in. concrete cylinders were measured while under sustained loads approaching the ultimate strength. Plotted data are shown to demonstrate that the change in the rate of plastic flow appears to be uniform up to three-quarters or more of the ultimate strength with an abrupt change at the "true" ultimate strength. Included are other data indicating that the plastic flow curve has the same general form independent of time and applied load leading to a new consideration in expressing factor of safety and developing beam theories on a factor of safety basis.

INTRODUCTION

A good fundamental expression for the plastic flow of concrete has been given by Straub[1] in the following form:

$$P = K \sigma^p t^q$$

in which P is the plastic flow deformation which is a function of the unit stress and the time t. The factor K is a parameter depending on the properties of the material. The exponents p and q are constants which depend on the properties of the material at the beginning of the stressed condition.

The author[2] attempted to combine the two terms K and σ^p into one and produced the expression

$$y = Cx^{1/a}$$

in which y is the plastic flow for a unit stress of unity, C is the coefficient used to involve both of the terms K and σ^p of the Straub formula, x is time in days, and $1/a$ is an exponent depending on quality of concrete, condition of curing, age at loading, etc. Both C and $1/a$ must be deter-

*Received by the Institute Nov. 15, 1948. Title No. 45-27 is a part of a copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 6, Feb. 1949, *Proceedings* V. 45. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than June 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

†Member American Concrete Institute, Asst. Director, Engineering Experiment Station, Ohio State University, Columbus, Ohio.

mined generally by experiment. By this formula the plastic flow for any given unit stress may be found by multiplying the value y by this unit stress. It therefore incorrectly, as it now appears, assumes that the plastic flow is directly proportional to the unit stress applied. A number of values of C and $1/a$ are given by the author in the earlier work[2]. His attempts to generalize on the variations of plastic flow with respect to degree of loading were not satisfactorily productive as this paper will show. Fig. 1 shows a variation in the coefficient C for various percentages of loading.

A thesis for Master of Science degrees by John B. Henry and Richard M. Gensert at Ohio State University in 1948 together with some determinations by graduate A.S.T.P. students in 1945[3] threw a little more light on the variations of C, particularly at very high loading.

The work by Henry and Gensert was undertaken to determine what its investigators preferred to call the true ultimate strength of concrete, or in other words, the ultimate strength for any length of sustained loading time. To accomplish this work they operated in two directions, one from instantaneous or very short time load-holding, as for standard compression tests, through a region of loadings which produced breaks after a

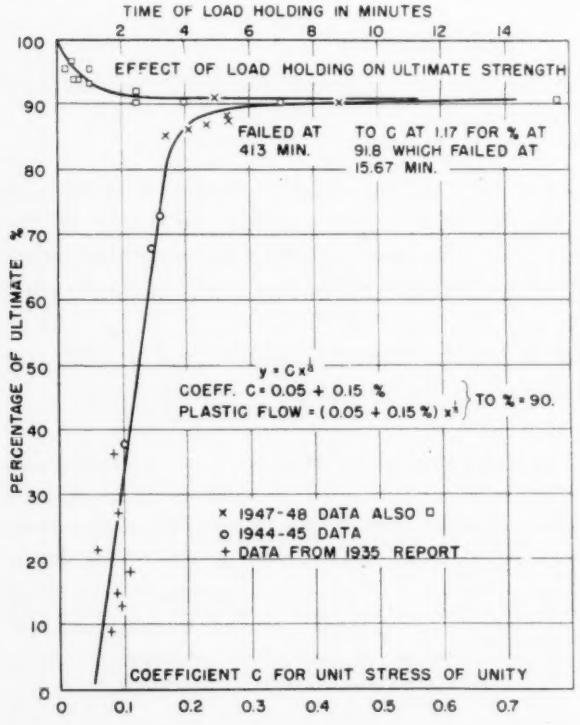

Fig. 1—Effects of load intensity and time on ultimate strength of concrete

time, to lesser loads that would not cause breaking, however long sustained, and the other by way of plastic flow tests leading up through the loadings at which the value of C becomes very large. In Fig. 1 their test results are indicated by ×'s and □'s, data taken from the work of 1945[3] by ○'s, and data on similar concretes from the earlier work[2] of the author by +'s.

TEST PROCEDURE AND RESULTS

The concrete used in this thesis work was all from one transit mixed batch made up of ¾-in. local gravel, local sand, and portland cement in the proportions: 1 cement: 2.40 sand: 3.32 coarse aggregate by weight as determined at the mixing plant. The ultimate strength (A.S.T.M.) was 5024 psi.

Thirty 6 x 12-in. cylinders were cast and given standard curing except for the last few days when they were dried out in room air preparatory to attaching resistance strain-gauges. The load was applied in a pendulum dynamometer type testing machine which had a load holding device capable of maintaining the load regardless of the deformation. Electrical strain-gauge readings were taken on a recording instrument. (Fig. 2).

Five of these 30 cylinders were used to determine the standard ultimate strength at 34 days age, the average of which, 5024 psi, served as a norm upon which to base percentages. Two were used to determine the modulus of elasticity using the method generally followed at Ohio State University, results were 4,600,000 and 4,550,000 psi. The stress-strain curve developed is a straight line up to 70 percent of the standard ultimate

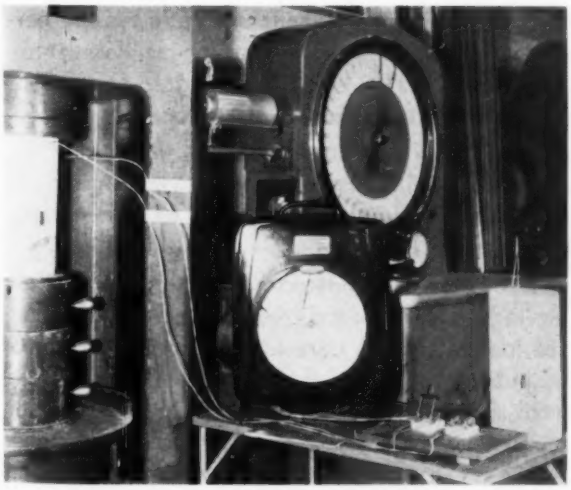

Fig. 2—Test equipment used to measure plastic flow

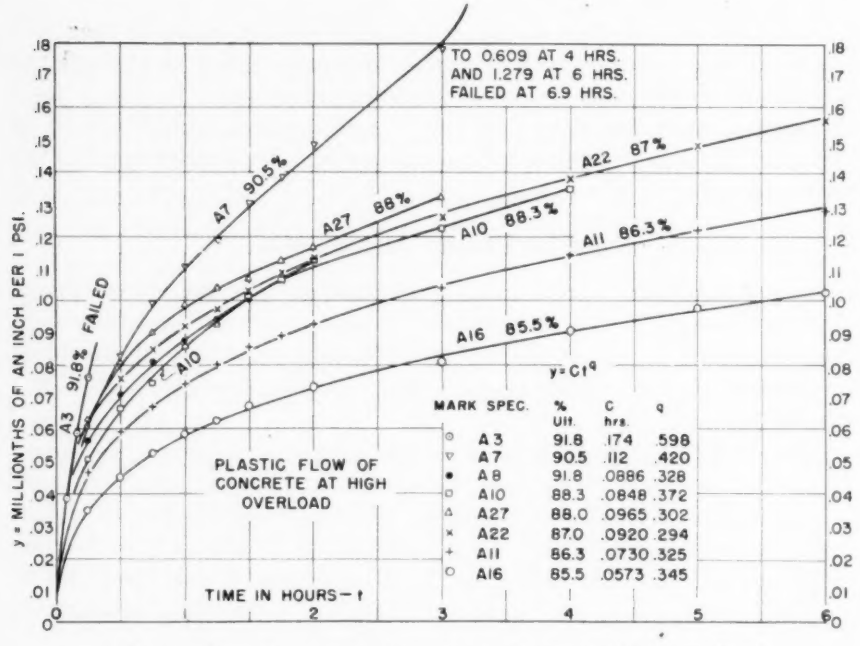

Fig. 3—Creep-time curves of concrete under sustained high overloads

strength[3]. Eight cylinders were used to determine the plastic flow curves shown on Fig. 3, and twelve for determining the time necessary to cause failure when the very high load was held for varying times. These were tested at ages from 33 to 54 days. Two were tested for compression at 60 days producing 5320 psi each. One of the 30 was bad.

It may be apparent that uniformity of strength and other characteristics were essential to the development of sufficient accuracy for the percentages. Three batches were tried but cylinders from one batch only were used for the data of this report. The A.S.T.M. Standard Deviation[4] for this one batch was 15 psi. and the Coefficient of Deviation, 0.30 percent.

On Fig. 3 are shown plastic flow-time curves for the 8 cylinders tested. Because of the shortness of time involved, it was thought to express time in hours, t, and the exponent is changed from $1/a$ to q.

The curves generally follow the same pattern as other plastic flow curves and when plotted on log-log paper show straight lines. The curve for cylinder A-7 which was loaded to 90.5 percent of standard ultimate strength, and which broke under this sustained load, shows a marked increase in its plastic flow from the proportionality of the others. Cylinder A-3 broke off short but A-7 held its load under very large deformations. The maximum value of 1.279, when translated into in. per in. for the

4550 psi load applied, amounts to 0.0058 in. per in. deformation, a deformation approximating a temperature contraction equivalent to 1000 F. The points shown on the curves for A-3 and A-7, when plotted on log paper, followed the straight lines very closely. The exponents for A-3 and A-7 are much greater than for the others for which the average of the $1/a$ values is 0.328, the general figure suggested earlier[2] for concretes in the working load range. This conformity permits the use of C to represent changes in the plastic flow rates to points well above working ranges.

PLASTIC FLOW AT NEAR-ULTIMATE LOADS

The data from the Henry and Gensert thesis serve to throw light upon the question of plastic flow at loadings near the ultimate. Fig. 1 shows two phases of this property of concrete. These data are plotted near the top of the lower curve. Data from the work of 1944-45[3] are plotted as circled points and from the publication[2] of 1935 as crosses. From this last source, only those concretes of similar elasticity, age and conditions of curing were selected.

It is of interest to note that the change in the rate of plastic flow seems to be uniform up to three-quarters or more of the ultimate strength (well through the working range), and that the change is abrupt near the "true" ultimate strength. Two separate groups of data confirm this. However, this exhibit of plastic flow rate change must be accepted as only a flash, indicating what might be, or possibly what is likely to be the situation, rather than what it is. If this rate change indicates the situation properly, then the curve equation for plastic flow may be stated without the expression σ^p, using for C or $K\sigma^p$ some such term as (0.05 + 0.15 percent) for all plastic flows through the working range.

The data at the top of Fig. 1 were obtained by loading the cylinders at an ordinary testing rate to predetermined percentages of the ultimate strength based on the norm of 5024 psi and holding the loads there until failure occurred; noting in each case the time elapsed from the time of obtaining the percentage load. The time scale is in minutes at the top of the figure. It seems apparent that the two curves should meet at the right since the failures indicated by the upper curve are seemingly due to excessive strains, as shown by the lower curve. The larger the coefficient C the greater the elastic, plus plastic strains in the concrete.

This bit of data indicates that the plastic flow curve is the same in general form for short as well as long times, and for high loadings as well as for low. It also suggests that the change in plastic flow for unit stress of unity for various loadings is a straight line function throughout all possible design loading ranges. It also indicates that the ultimate strength of concrete must be reduced by an appreciable amount when the

question of time is involved, as in most practical situations. This means a new consideration when expressing factor of safety and developing beam theories on a factor of safety basis.

The percentage representing the "true" ultimate strength may be expected to fluctuate both above and below the 90 percent point. The third batch made for the thesis work showed 85 percent as the location of the "true" ultimate strength.

CONCLUSION

Considerably more work should be done to confirm, or prove false, this straight line variation in the change of plastic flow for unit stress of unity. More data should appear in the middle ranges. Also, the "true" ultimate strength for lean concretes, concretes of special aggregates, concretes cured wet vs. dry, concretes of wet consistency vs. dry concretes, concretes vibrated into place vs. puddled concretes and concretes containing air-entraining agents might profitably be investigated.

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In September **Letters from Readers** replaced **Job Problems and Practice,** with broader scope, more flexible form, and we hope more adaptability to the thoughtful, busy reader's use, as a forum for opinions, experiences and inquiries within the field of ACI activity. A letter does not need the studied treatment of a major paper to present valuable ideas for ready access to kindred minds.

Letters from Readers*

BY WAY OF SYNOPSIS

C. MARTIN DUKE describes tests devised to determine usability of cement stored out-of-doors in Guam.

Cement balls in concrete are attributed to improper mixing action by R. H. BAUMGARTEN

Deterioration of Sacked Cement (LR 45-13)

C MARTIN DUKE recounts his Pacific island experiences testing partially hardened cement.

Some information on the deterioration of sacked cement subject to tropical exposure has been obtained from experiences with cement stored at Guam, Marianas Islands, latitude 13 degrees north, longitude 145 degrees east.

About July of 1945 a quantity of A.S.T.M. Type I portland cement was delivered at Guam in paper sacks and placed in open storage. Eighteen months later, at the time of testing, a goodly portion of this cement was still unused. In the interim it had been frequently deluged during two of Guam's 88-in. (average) rainy seasons, and had been exposed to an average relative humidity of 80 percent and an average air temperature of 84F. Some of the cement was covered by tarpaulins during an unknown portion of the 18 months, though in August of 1946 there were very few tarpaulins in place. In the storage yard the cement was stacked about 10 ft. high on 4 x 4 ft wood pallets, with

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1-ft horizontal spaces between the pallets. Considerable speculation arose as to whether a portion of the cement might be reclaimed and used in construction; as a result the following steps were taken by the Guam Testing Laboratory of Pacific Islands Engineers, Architect-Engineer.

A survey to determine the extent of deterioration of the cement was made by systematic inspection which involved probing with a sharp instrument and cutting open occasional sacks. Degrees of hydration based on this inspection were assigned, as shown in Table 1. It was estimated that none of the cement was of Degree A (practically unweathered), 10 percent was of Degree B (containing easily-broken lumps), 50 percent was of Degree C (end portions hardened), and 40 percent was of Degree D (sack completely hardened).

Five samples of the cement, respresentative of the various brands, conditions of exposure, and visual degrees of hydration, were subjected to laboratory ignition-loss and mortar-compressive-strength tests according to A.S.T.M. C114–46 and A.S.T.M. C109–44, respectively.*

TABLE 1—TESTS ON DETERIORATED CEMENTS AT GUAM

Brand[a]	Condition of exposure	Visual inspection		Test results	
		Degree of hydration	Description	Ignition loss[b], percent by wt	Mortar compressive strength at 7 days[c], psi
Lehigh	Under tarpaulin, very little exposure	B	Sack soft throughout except for easily broken lumps	1.8	2750
Lone Star	Bottom of pile	C	End portions hardened—center soft with breakable lumps[d]	3.1	1800
Blue Diamond	Top of pile, semi-exposed	C	End portions hardened—center soft with breakable lumps[d]	2.3	1800
Monolith	Top of pile, semi-exposed	C	End portions hardened—center soft with breakable lumps[d]	3.0	2050
Lehigh	Top of pile, extreme exposure	D	Sack completely hardened. End portions harder than center portions[e]	8.0	350

a. All cements were A.S.T.M. Type 1.
b. By A.S.T.M. C114–46. A.S.T.M. specification C150–46 permits 3 percent maximum.
c. By A.S.T.M. C109–44 except as noted in text. A.S.T.M. specification C150–46 permits 1800 psi minimum.
d. Tests made on center portion, pulverized with hands.
e. Tests made on center portion, pulverized with mortar and pestle.

*The latter was not complied with in its entirety in that the molds were of oiled wood and the Ottawa sand was 20–30 mesh, owing to lack of proper materials at the time. Resultant errors were shown to be unimportant by comparison with results of control tests on a new cement.

Results of the tests are shown in Table 1, from which it may be inferred that: (1) correlation among ignition loss, mortar compressive strength, and visual degree of hydration is very good; (2) cement protected from rain will still meet A.S.T.M. specification C150–46 after 18 months open storage on Guam; (3) cement lumps of this age which can be pulverized with the hands will, when pulverized, come close enough to meeting A.S.T.M. specification C150–46 to justify use of such cement in ordinary concrete work; (4) cement which is so hardened as to require grinding is not suitable, even after grinding, for any cementing purpose.

Based on the survey and laboratory tests it was estimated that approximately half of the remaining cement could be reclaimed by proper processing, possibly involving grinding. Raymond E. Davis was then commissioned by the construction contractor to make independent laboratory tests and to recommend a procedure for reclaiming. Mr. Davis tested ten representative samples at the Engineering Materials Laboratory, University of California, Berkeley, which essentially substantiated the inferences of the above paragraph and in addition indicated good correlation of potential heat of hydration and of sack weight with ignition loss, mortar compressive strength, and visual degree of hydration. He found that cement samples with an ignition loss of 8 percent or less were usable, in that they would give a 28-day mortar compressive strength usually in excess of 2000 psi. He estimated the cost of reclaiming equipment, including ball mill, at about $30,000, with about 20 months required for delivery. Analysis by the construction contractor indicated that total reclaiming costs would be so high that it was resolved to abandon the plan.

The work described herein was under the cognizance of Bureau of Yards and Docks, Rear Admiral J. J. Manning, C.E.C., U.S.N., chief of the bureau, Captain E. L. Hansen, C.E.C., U.S.N., officer in charge of construction, Bureau of Yards and Docks Contracts, Marianas Area, and Mr. P. C. MacCallum, general manager, Pacific Islands Engineers.

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Cause and Cure for Cement Balls in Concrete (LR 45-14)*

R. H. BAUMGARTEN, London, England, analyzes mixer action.

If concrete is mixed in a properly designed machine there can be no lumps or balls of cement in the mix, except when the cement used is old and contains hard lumps.

The trouble of which Mr. Rader complains is caused by the failure of many concrete mixers to perform their job thoroughly. In hand mixing, if the worker only turns the materials over and lets them fall off the shovel he will produce a mix that is far from perfect. If, on the other hand, he strokes with the back of his shovel over the material, pressing the shovel down, every time he has turned a shovelful over, he squashes any lumps of cement that may be in the mix. This effect can be produced in a mixing machine only if the latter is provided with a roller, or rollers, which produce the same results as the pressing down and stroking of the back of the shovel. As far as I know the only machines of this type that are built are the German "Eirich", the British "Pegson", "Cumflow" and "Winget" and the Danish "Universal" mixers.

*See also ACI Journal October 1948, p. 181; November 1948, p. 261; December 1948, p. 345.

Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

Ornamental concrete

E. Roelofsen, *Ingenieur* (Utrecht), V. 60, No. 6, 1948

Ceramic Abstracts
Oct. 1948 (Max Hartenheim)

The dead, colorless appearance of concrete does not lend itself easily to ornamental decoration. Means and methods of adapting concrete for ornamental purposes are discussed. This type of cement usually consists of a core of ordinary concrete covered with an outer layer of portland cement with an aggregate of granite, quartz, prophyry, basalt, and a special kind of colored slag. The surface of the outer layer is ground and polished to give it a more pleasing appearance. The elements for the covering layers must be weather resistant and abrasion resistant; the formed parts must have exact dimensions, and the coloring must fit in with the whole building style. Ordinary portland cement is used for dark layers, and white portland cement for lighter layers. The loss of material with different aggregates was determined under the (Dutch) standard test N502.

Aggregate (0 to 3 mm)	Sliding test (mm)	Heavy sandblasting test (g.)
Quartz A	1.3	18.5
Quartz B	2.2	21.5
Quartzite	2.4	27.0
Granite A	1.9	17.0
Granite B	1.9	25.0
Granite C	2.0	18.3
Granite D	2.0	16.5
Basalt	2.3	22.0
Porphyry of Quenast	2.6	19.6
Comblancien	4.2	26.5
Travertine	3.4	22.5

The colors used in this material must be insoluble and resistant to weather, to attack by lime liberated in setting and to the action of light; the pigment must be finely ground, thus increasing the intensity of the color. The coloring substance must not contain anything that disturbs the setting or the strength of the concrete. A comparison is made between the properties of American ornamental concretes and Dutch standards

*A part of copyrighted Journal of the American Concrete Institute, V. 20, No. 6, Feb. 1949 *Proceedings*, V. 45. Address 7400 Second Boulevard, Detroit 2, Mich.

Precast concrete units simplify construction of west coast marine structures

BEN C. GERWICK, *Civil Engineering*, V. 18, No. 7, Aug. 1948, pp. 46-50

Reviewed by J. R. SHANK

A pier structure 3800 ft long and 26 ft wide at Point Richmond, California was constructed on concrete piles all parts of which were precast at a site 22 miles away except the interiors of the pile caps and the curbs.

The piles were cast in special forms so that a tier as many as ten pile widths high could be poured at one time. The concrete was vibrated into place. The horizontal joints between piles were made by a coat of emulsified paraffin. A gantry crane running on tracks on pile trestles picked up the piles and loaded them onto barges for transportation to the construction sites.

The pile driving had to be very accurate. Concrete pile cap shells or forms with 3½-in. walls were precast and transported to the site and placed on temporary steel channels resting on timbers clamped to the piles. Anchor bolts in the tops of the piles extended up into the insides of these shells through holes in the bottoms over the pile tops. After these cap shells had been accurately placed, reinforcing steel assemblies were placed in them and concrete was poured into the hollow space around this reinforcing steel to the pile tops, thus erecting a perfectly placed concrete cap on concrete piles without forms.

These reinforcing steel assemblies carried anchor bolts for anchoring the slabs over the caps. Three-in. pipe sleeves were used in the slabs to contain these anchor bolts. After setting, non-shrinking grout was poured into the pipe sleeves to fix the anchor bolts and to grout in the bearing. Strips of asphalt-impregnated fiber were laid around the edges of the caps to serve as supports for the slabs and dams for this grout. The joints between the slabs were grouted with a thin grout. Curbs were poured in place and a 2-in. asphalt concrete pavement was laid as a wearing surface. These slabs, all precast 22 miles away were 20 ft by 6 ft. 7 in. wide and weighed 13 tons. As many as 60 of these slabs were set in one 8-hour day by one crew.

Measurement of the heat of hydration of cements by the thermos method (La mesure de la chaleur d'hydration des ciments par la methode thermos)

J. CLERET DE LANGAVANT, *Centre D'Etudes et de Recherches de L'Industrie de Liants Hydrauliques*, Publication Technique No. 7, July 1948

AUTHOR'S SUMMARY

Calibration of the Dewar testing instruments, used for the thermos method, must be done by observations under steady conditions (calibration cylinder electrically heated). The coefficient of loss may be adjusted to obtain by the thermos method the same values of hydration heat as by the dissolution method.

The method is reliable and very simple to use. When used with an air temperature control meter, it permits the determination of hydration heat when the cement is submitted to arbitrarily chosen heating conditions. Therefore, adiabatic instruments become less useful.

The test can be used as a rapid control of cements on delivery.

Prestressing applied to strengthening a church tower in Staffordshire

MAGNEL, *Concrete and Constructional Engineering*, V. 43, No. 9, Sept. 1948, pp. 281-283

Reviewed by GLENN MURPHY

Application of the wedges, sandwich plates and jacks devised by Professor Magnel to the strengthening of a tower are described. The method consists essentially of constructing a prestressed reinforced concrete beam within the masonry wall. Photographs and drawings are included.

Experimental determination of the composition of concrete mixes

MAURICE BUISSON, *Annales de l'Institut Technique du Batiment et des Travaux Publics*, Circ. No. 21, Nouvelle Serie, May 1948 Reviewed by G. L. BLANCHARD

The method is based on the fundamental laws evolved by Feret. The author calls *Factor of Resistance* the ratio of the volume of cement to the sum of the volume of water and the volume of the voids. This ratio is characteristic of the compressive strength which varies proportionally to the square of its value. For practical purposes the ratio may be expressed as a proportion by weights.

After a theoretical discussion, the author describes the procedure which involves weighing the materials of the assumed mixes, a succession of elementary computations and the compaction of the mixes in a specially designed mold. The selection of the optimum mix requires numerous operations as cement, aggregates and water may vary in proportion to each other. Curves plotted in each case depict the influence of the variation of each component on the *factor of resistance*. Its maximum value for a mix obtained by interpretation of these curves appears on a final diagram where the percentage of cement and the proportions of sand and gravel are kept constant with the *factor of resistance* varying with the water content.

The author remarks that, from the data collected, the *factor of resistance* and its components varies considerably depending on the nature of cement employed because the coefficient of friction of the cement is not a constant. Therefore, a separate study is necessary for each cement used.

Personnel can be rapidly trained to use this method, and it provides a sound guarantee against risks of dangerous or wasteful empirical procedures.

Expansion of cements measured by autoclave tests (L'Expansion des ciments mesuree par l'essai a l'autoclave)

LEON BLONDIAU, *Centre D'Etude et de Recherches de L'Industrie des Liant Hydrauliques*, Publication Technique, No. 10, Nov. 1948 AUTHOR'S SUMMARY

The hydration of cement is too slow a process for the cement to reach the final hardening soon after wetting. The attendant changes in volume are accordingly delayed and therefore the more dangerous. The variation in volume is due to a number of factors, some of which are directly dependent on the cement, *e. g.* magnesia, free lime and tricalcium aluminate. The expanding action of these varies according to whether they are dissolved when the clinker is vitreous or in a crystalline state; they are also strongly influenced by the temperature and duration of firing. To determine the time factor in the action of these components a number of tests were designed, including one using prisms of pure cement paste treated with steam superheated in an autoclave. On this an American Committee on the stability of cements has worked extensively and it is prescribed as an admittance test by the A.S.T.M. The work of Young justifies imposing such a severe test because it eliminates cements which tend to behave abnormally in normal conditions of use. The Societe Anonyme des Ciments de Thieu have carried out research into the factors affecting the manufacture of autoclave-stable cements. It has established that the composition of the clinker must be regulated so that the magnesia content is kept below 1 percent; the tricalcium aluminate content (calculated by Bogue's formula) between 10 and 11 percent, and free lime at below 1 percent. A fineness of 2250 sq *cm/g* (Wagner), facilitates the production of a stable cement. Lastly, proof is given of the perfect stability of blast-furnace cements, even when the magnesia content reaches 13 percent, and of the increase in stability of portland cement by the addition of slag and pozzolans.

A rapid method of testing materials for the alkali-aggregate reaction

D. O. Woolf and T. R. Smith, *Public Roads*, Dec. 1948 Authors' Abstract

A discussion of the most common methods for testing materials to determine their susceptibility to reaction with the alkali in cement, and the development and use of the Mason jar method which is considered suitable for the detection of reactive materials in a period of 30 days or less.

In the Mason jar method, a mortar or concrete is prepared with the aggregate under test and a high alkali cement. The mixture is placed in the jar and the latter capped. After 24 hours, about 25 ml. of water is added to the jar, the cap replaced securely, and the jar placed in storage in an inverted position. The jar is examined daily until multiple cracking appears, or until a suitable age is attained without cracking.

To prevent reaction between the glass of the jar and the alkali in the cement, the inside of the jar is coated prior to use with an alkaliproof paint. A number of coating materials have been found satisfactory for this purpose, but the best appears to be a pigmented material with a chlorinated rubber base.

Most of the samples tested have been stored in room air, with the temperature from 70 to 80 F. Storage at 100 to 110 F appears to accelerate the reaction to an appreciable extent. At elevated temperatures, the storage should begin immediately after the preparation of the jar to prevent cracking due only to the thermal expansion of the hardened mortar or concrete.

This method is considered of most value for the preliminary separation of aggregates on the basis of their relative susceptibility to reaction with the alkali in cement. Aggregates which are highly susceptible to this reaction, and aggregates which are relatively non-reactive, can be detected readily. The test can be made in the field with simple equipment, and the materials used may be measured by volume if a balance is not available.

Durability of concretes and mortars in acid soils with particular reference to drain tile

Dalton G. Miller and Philip W. Manson, *University of Minnesota Agriculture Experimental Station*, Technical Bulletin No. 180, 1948 Reviewed by E. C. Wenger

This bulletin describes the results of tests on experimental concrete drain tile and 2 x 4-in. concrete and mortar cylinders installed during the past 25 years in Minnesota and Wisconsin peat and mineral soils, ranging in *pH* from 4.1 to 8.00.

In the manufacture of the tile and the test specimens, two aggregate gradings were used with four mixes of proportions 1:1, 1:3, 1:5 and 1:7 by volume, and 20 different brands of cement of which four were special cements. Six admixtures, two surface treatments and different curing conditions were also investigated.

The results indicate that practically all the concrete pipe and specimens showed evidence of corrosive action after 20 years exposure in peat. The degree varied with the acidity of the peat and the unit strength of the concrete. Poor quality concrete made of leaner mixes gave unsatisfactory service. Rich mixes of high strength and low permeability gave the greatest resistance to action of soil acids. The investigation indicated that drain tile installed in peat soils must be at least of the quality required of Extra-Quality Drain Tile by the Standard Specification for Drain Tile—A.S.T.M. designation C4-24.

None of the admixtures used in the tests improved the resistance of the concrete to the action of soil acids. The chemical composition of the cements used had little effect on resistance, except that the special high alumina cements were slightly more resistant than the normal portland cements. The only special procedures in manu-

facturing the tile which improved the resistance of high strength concrete were steam curing at 345 F., and bituminous paint or linseed oil coatings.

Extensive tables show the results of the various tests made on the concrete tile and cylinders.

Slender reinforced concrete columns

J. M. Prentis, and A. D. Ross, *Concrete and Constructional Engineering*, V. 43, No. 9, Spet. 1948, pp. 261-270 Reviewed by Glenn Murphy

The authors investigate the effect of creep upon the load-carrying capacity of reinforced concrete columns. With the inevitable eccentricity present in columns a non-uniform stress situation exists and creep occurs more rapidly on the concave side thereby increasing the eccentricity and accentuating the non-uniformity of the stress. The rate at which this action continues is first estimated by assuming that the rate of creep is proportional to the stress. The authors report as simpler a method in which E is replaced by an effective modulus which is a function of time. The second procedure gives a higher strain than the first method, with the true result expected to lie between the two. Curves are shown comparing the computed loads for failure after infinite time using the two procedures. For large values of slenderness ratio the eventual strength is less than one-third of the strength in a short time test. Analyses are given for plain concrete as well as reinforced concrete columns.

Determination of the grain size of cements (determination de la granulometrie des ciments)

Ch. Momot, *Centre D'Etudes er de Recherches de L'Industrie de Liants Hydrauliques*, Publication Technique No. 8, July 1948 Author's Summary

This paper reports a study of the Wagner turbidimeter and the Pearson and Sligh flourometer in an endeavor to establish a relation between their indications and physical reality.

Both devices give the percentage by weight of grains below a given size. However, these two figures are not always in full agreement.

A process has been developed for counting under the microscope with a high degree of accuracy, which data is used to draw the granulometrical curves from 15μ. Several cements previously tested with the turbidimeter and the flourometer were retested and figures obtained were compared for grain sizes below 15, 30 and 45μ respectively.

A study of the results showed that the flourometer is always in agreement with the microscope while the turbidimeter is distinctly inaccurate for certain cements. This inaccuracy of the turbidimeter may be because c is not rigorously constant in the formula $S_d = c\ (\log I_o - \log I_d)$.

Two good means are available to determine the granulometry of a cement more quickly and with the same physical significance as with the microscope:

1) Measurements at the three nozzles of the flourometer giving the segregation to 15, 30 and 45μ.

2) The granulometrical curve of the turbidimeter which is quite similar to the curve obtained with the microscope and from the result of the 30μ segregation with the flourometer locate this curve in height, since the turbidimeter sometimes gives a false position.

Results obtained with the permeabilimeter cannot be related to those given by the above devices. This apparatus may be used as a detector but cannot take the place of one of the devices mentioned above to compare various cements.

The hydrolysis number in cements

A. REBUFFAT, *Chemie et Industrie*, Jan. 1948 AUTHOR'S SUMMARY

The constitution of cement is commonly thought of as being fully resolved, but the established scheme does not suit all the cases. In 1934 the author observed that it was possible both to reduce or to oxidize all the iron contained in portland cements without any change in the characteristics of the clinker; proof of the iron being contained by a silicate. By hydrolysis with bromine water and afterwards with hydrochloric acid a *Hydrolysis number* was determined which is a parameter, constant for each cement, corresponding to the maximum of bases that in a given cement are set free by the hydrolysis. All cements during the process leave residues composed of clear and colored fractions having the zeolitic property of changing the bases. Examining the colored part of the residue in various type of cements, the author identifies three groups:

1. Portland and pozzolanic cements (clinkers)—calcium silicoferrite which is a compound of silica and dicalcium ferrite.
2. Slag clinkers—a mixture of silicoferrite, as in the first group, with the residue of the hydrolysis of the calcium aluminoferrite.
3. Iron clinkers—an unhydrolyzed mixture of dicalcium ferrite, silica, and tetracalcium aluminoferrite.

Reduction of cements with hydrogen demonstrates that it is a mistake to assume that the alumina is contained in the cements as a tetracalcium aluminoferrite, whereas the iron is normally in the clinkers as a dicalcium ferrite mixed with small quantities of silica.

Chemical analysis of the clear part of the residue shows it to have a zeolitic composition. In the portlands these zeolites seem to be of the scolecite type; in the pozzolanic cements are found the chabazite or gmelinite type, and in the slag cements the philipsite. What is really important is the formation of a zeolite and not its type.

The setting of cements is nothing but a phenomenon due to the formation of zeolite. If, during its formation, the cement grains are in contact they stick together forming a sort of continuous reticulum meshing all the other components. If the grains are separated the zeolite forms all the same, but the cement does not set. The speed of setting is certainly related to the percent of zeolite that can be formed and consequently to the quantity of aluminosilicate of calcium.

The pre-stressed concrete runway of Paris—Orly international airport

M. NETTER, *Annales de l'Institut Technique du Batiment et des Travaux Publics*, Circ. No. 5—Nouvelle Serie—Jan. 1948 Reviewed by G. L. LANCHARD

A short introduction by M. R. Freyssinet states the basic conditions of the problem of rigid runways and explains the reasons for the adoption of the system of pre-stressed slabs employed. Aware of the technical difficulties of the problem of rigid runways, and owing to the acute shortage of steel and cement in France, an experimental runway of prestressed concrete was used to conserve materials.

Plain concrete slabs were laid on the foundation in checkerboard fashion. Cables were placed between the standard slabs and passed through specially designed longitudinal curb slabs into which they were anchored.

An ingenious device secured the necessary longitudinal stress. A line of frictionless joints at 45 degrees runs across the whole length of the 400 m. runway dividing it into 6 triangles of 120 x 60m and 2 of 60 x 60m. At each end a rigid abutment embedded into the ground prevents longitudinal movement of the slabs. Tension applied to the cables compresses the slabs in the transverse direction. At the same time, through

the action of the joints at 45 degrees the slabs tend to expand longitudinally but the rigid end abutments prevent this expansion and this counteraction creates the required longitudinal stresses.

The author discusses the behavior of the slabs under load and temperature action, together with some data on the measurement and computation of stresses. Previous to the adoption of the project a series of extensive tests were performed on an experimental prestressed slab 14 x 12.50 x 0.16 m. After the completion of the experimental runway, typical sections or points were submitted to field tests which included deflection measurement under statically applied load and repeated dynamic action of wheels. No cracks appeared on the top face of the slabs submitted to tests. Data are given as to the ratio of pressure to depression and distribution of loads on the subgrade together with photographs and sketches of the construction.

Clark dam, Tasmania

H. H. Thomas, *Journal of Engineers*, (Australia), V. 20, No. 9, Sept. 1948 Reviewed by J. R. Shank

Clark Dam is an arch-gravity dam 200 ft high on the Derwent River at Butler's Gorge, Tasmania. The maximum thickness of the base at crown-center is 80 ft. Radial contraction joints were spaced about 50 ft as measured on the upstream face. The joints were grouted and cooling pipes were built into the concrete.

Low heat concrete was used on the thick lower parts; half low heat and half normal portland cement were used on the upper works. This blended cement was found to develop 126 BTU per lb at 7 days and 133 BTU at 28 days. The alkali content was held to not more than 0.6 percent, although the aggregate was found to be a "safe" aggregate with ordinary cement. The aggregate was said to be of "dolerite" found locally. The fine aggregate was crushed so as to be made up of particles approaching cubes in shape, but no effort was made to eliminate flat or elongated particles in the coarser aggregates. The proportion of cement was 10.1 percent of the mix by weight and the aggregate from No. 4 mesh to ¾-in. was limited to 8 percent. The concrete weighed as much as 167 lb per cu ft.

Pipes, ¾-in. diameter for artificial cooling, were spaced 5 ft vertically by 5 ft 9 in. horizontally. Natural river water which ran as low as 40 F, was circulated. Some 150,000 ft of piping was embedded and water was circulated at the rate of 1.5 mgd by means of four 4-in. centrifugal pumps. Grouting of expansion joints was done at a temperature of 42.5 F, which is 2 F below mean annual temperature. Electric gap measuring devices were used at the joints.

The concrete was poured in 5-ft lifts from two 10-ton cableways on head and tail towers moving on parallel tracks. Blaw-Knox 4 cu yd buckets were used.

The article which covered much more than the concrete work contains maps, data curves, plans, and details of construction.

Tests on concrete columns reinforced with square twisted steel and mild steel

K. Hajnal-Konyi, *The Structural Engineer*, V. 26, No. 7, July 1948, pp. 395-431
Reviewed by Glenn Murphy

The investigation reported by Dr. Hajnal-Konyi was to determine the behavior of concrete columns with ordinary ties reinforced with single twisted bars of B.S. 1144, having a proportional limit of about 50 ksi and no yield point. The author establishes a condition for maximum load which a column will carry by differentiating the expression for load and equating the derivative to zero. This leads to the condition that

at maximum load and for equal strain in the steel and concrete, the area of the concrete times the slope of the tangent to the concrete stress-strain diagram must equal minus the area of the steel multiplied by the slope of the tangent to the steel stress-strain diagram at the same strain. This means that if the concrete reaches its ultimate strength at a strain less than the strain in the steel at its yield point, the maximum load on the column will be reached after the stress in the concrete has passed the ultimate. In this development the author assumes that the concrete will not disintegrate at the ultimate, but that it is capable of sustaining load for strains greater than the ultimate.

On the basis of the analysis a series of tests was conducted on columns 8 in. square and 4 ft long. The series included two percentages of reinforcing with both mild steel and high-tensile steel and two strengths of concrete—2250 psi and 5000 psi. Strain records were taken throughout each test to failure. The test results indicate that the maximum measured strain in the columns containing the high-strength steel was substantially greater than the maximum strain in the corresponding columns containing the mild-steel reinforcement. The ultimate loads agreed closely with the loads computed by the author's theory.

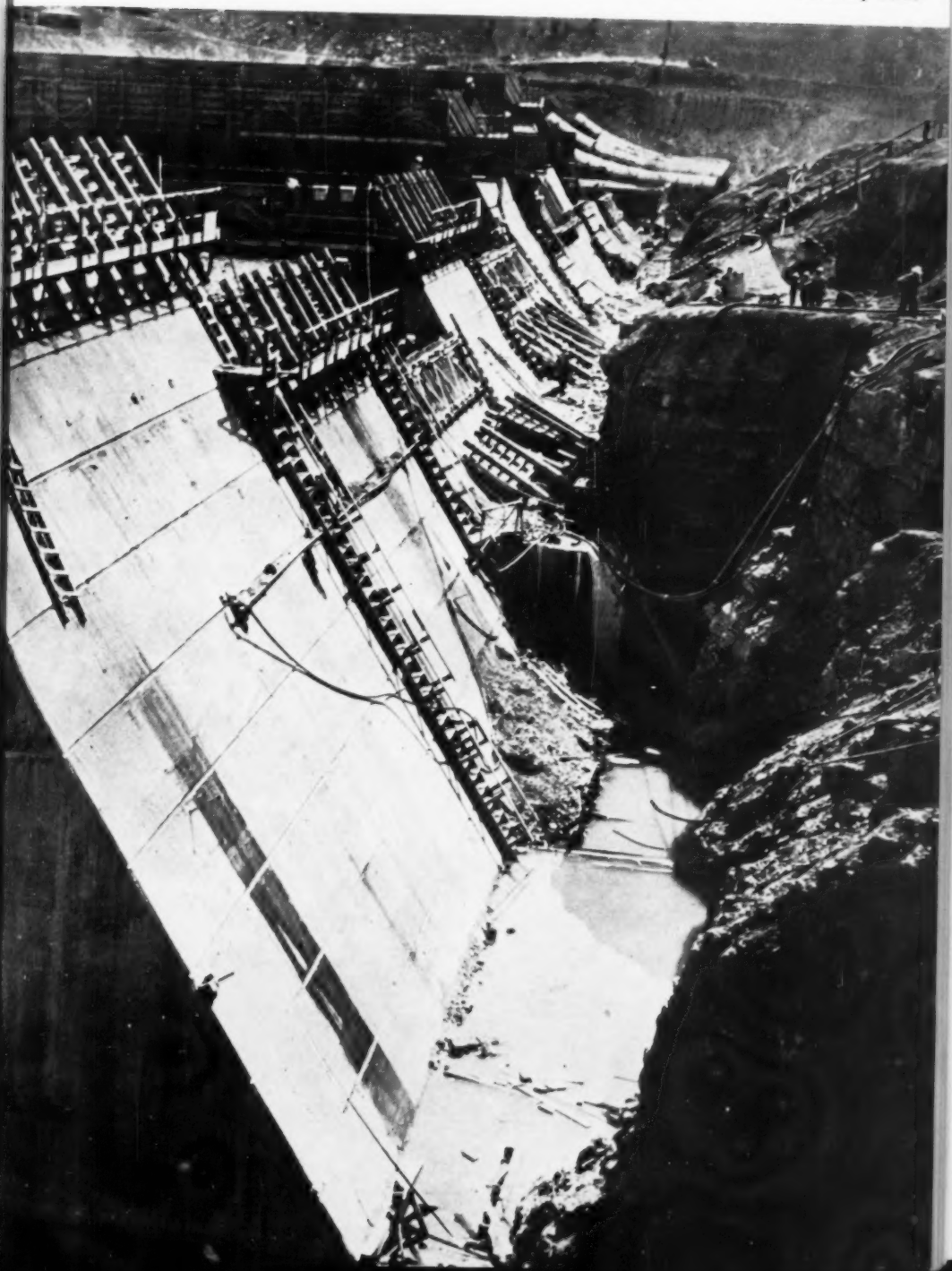
The author discusses his results with reference to the British Code of Practice and makes suggestions for improvement of existing design formulas.

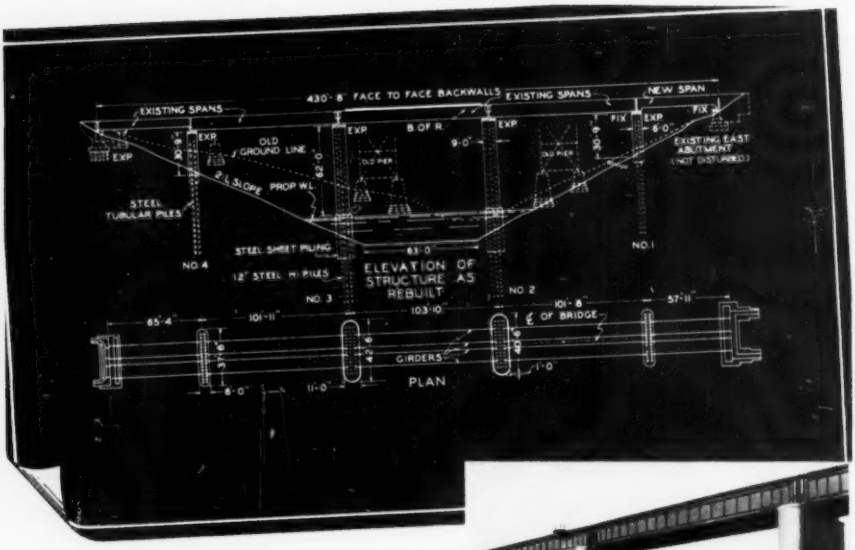
He indicates that with high-tensile steel the factor of safety is increased if creep occurs in the concrete because load will be transferred to the steel which has reserve strength. Other advantages indicated for the columns containing high-tensile steel are increased strength with neither increase in dimensions nor increase in concrete strength, comparatively high toughness, and warning by cracking before ultimate load is reached.

This paper should be of interest to all concerned with design of reinforced-concrete structures.

ACI News Letter

ol. 20 No. 6 JOURNAL of the AMERICAN CONCRETE INSTITUTE February 1949





General view of bridge. Note H-piles in base of unfinished pier No. 3.

PREPAKT REPORTS . . .

TOP ECONOMY IN BRIDGE RESTORATION

When it became necessary to reconstruct a double-track bridge to accommodate deepening and widening of the power plant raceway channel beneath it, Canadian National Railways gained many advantages with a pier design of steel H-piles encased in Prepakt Concrete, and by using Intrusion-Prepakt methods for the work.

As shown in the print above, the steel H-piles were driven deep to bedrock and the new piers were built underneath the existing spans and in close proximity to the existing foundations.

Each pier is composed of a group of steel H-piles, extending nearly the full height of the pier, then enclosed within PREPAKT concrete so that a monolithic pier shaft is formed. The strength and lateral rigidity of this design permit a relatively slender pier.

This is another example of construction economy where PREPAKT methods made possible the entire reconstruction of this bridge on the original alignment and without interruption of rail traffic. The new design and piers even made possible the re-use of the existing spans!

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ACI

News Letter Contents

February 1949

On the cover—general view of monoliths under construction at Wolf Creek Dam, Kentucky. "Cantilever" type steel forms are used which eliminate 60 to 75 percent of anchors. Forms are anchored into blocks of concrete previously poured.

Blaw-Knox photo

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Who ARE *and What* IS *the American Concrete Institute*

Perhaps it's an odd question to originate in ACI headquarters, but a very important one, and it springs from the fact that the answers are near. ACI's objectives are about as clear as they should be, as stated in our Charter—broad enough within the compass of the mandate of those objectives, to permit a sensitive ACI administration to aim to serve the needs of members from one period to another. We must continue to organize the efforts of ACI members in a non-profit public service in, as the Charter says, "gathering, correlating and disseminating information for the improvement of design, construction, manufacture, use and maintenance of concrete products and structures"—but with what special objects within the Charter's framework?

A few months ago we launched a kind of plebiscite; asked all members in the United States (and the questions will go to members elsewhere shortly) to tell us what they *do*—what the members' employers *do* and what others as employees *do* for their employers. This was on the assumption that the professional and business activities of members should shape the key that unlocks their interest.

Response in general was prompt and cooperative and the membership department is staggering under the load of detail involved in the creation of a new and additional list of ACI members which sorts members into their respective activity categories. It is to be, we hope, a means of knowing more precisely your professional activities and your specializations with all these categories appraised numerically—the democratic way—not to the end that the majority shall rule ACI affairs but to gage the needs of important minorities. The

plan is to make the classification the means of more contacts with groups having homogeneous interests for a clearer idea of what ACI members as individuals expect of ACI as an institution.

It happens that at the break of World War II the Institute had been climbing slowly back from a membership much diminished in the depression thirties (the low was 1154 in 1934) to 1909 members on January 1, 1944.

January 1, 1949 membership stood at 4,395—a gain of 130 percent in the five-year period.

In October last the Board adopted "goals" (see News Letter, November 1948, page 3). One of the goals was a doubled membership. How? That was the question before headquarters staff. We look now to the returns on the plebiscite for at least an important partial answer.

While ACI was making a five-year gain of 2,486 members, that net gain was the difference between new members added and old members lost by death, by resignations and for non-payment of dues. Thus about two-thirds of our members are new within a five-year period. What do they individually and collectively look to ACI, the institution, to give them?

It has been said by ACI critics, even by some of its past presidents in their retiring addresses, that ACI is or has been too academic. From a careful study of the answers in what is believed to be a representative "sample" of ACI's recent questionnaire returns, it appears that the Institute is certainly not perceptively academic in its membership activities or in what it expects from ACI's publications.

As a starting point in setting up membership classification on the basis of the sampling of alphabetized returns, 260 forms from members A to B inclusive (Abbett to Burmeister) were examined—college professors, contractors, construction superintendents, inspectors, concrete control engineers, sales engineers, directors of and technicians in research; architects, structural engineers, highway engineers, materials engineers; producers of: masonry units, admixtures, construction machinery, cement, aggregate, ready mixed concrete, steel reinforcing and many other items, looking to ACI for further development for better structures.

We know, as we have long suspected, that the ACI membership is far from being as homogeneous in character as we expect of a good concrete mixture. Members have but one common ground of interest—concrete; they want to know how to use it in design, in construction, in manufacture so that hundreds of private purposes in its use may best be served.

These paragraphs are being written that you may share the interest that ACI headquarters feels in this study of the working needs of ACI members and a staff effort to serve them.—Harvey Whipple.

Forecast of 45th ACI Convention

Hotel Statler, New York, Feb. 23-25, 1949

As the 45th Annual ACI Convention—peak of activity for the ACI year—approaches, an excellent technical program is clearly evident. In addition to the papers announced and briefly synopsized in the January News Letter competent discussion is assured, thereby adding to the vitality and interest of the program.

B. D. Tallamy

Concrete pavements

On **Wednesday afternoon** at the first technical session, B. D. Tallamy will present "Control of Scaling of Portland Cement Concrete Pavements Resulting from the Use of Chloride Salts." Charles M. Noble will follow with "Experience with Air-Entraining Concrete in New Jersey." Several discussers have assured TAC of their participation.

Air entrainment

The two papers on air entrainment by J. F. Barbee and by R. F. Blanks and W. A. Cordon are bound to draw comments from many on experiences.

Two-way slabs

The **Thursday morning** session, February 24th, will be of particular interest to architects and structural designers. "Rational Analysis and Design of Two-Way Concrete Slabs," by C. P. Siess and N. M. Newmark, University of Illinois, may well affect future action of ACI Committee 318 in revising the method of flat-slab design now specified in the ACI Building Code. The Siess-Newmark paper was published in the December ACI JOURNAL.

J. F. Barbee

C. P. Siess

Footings

Also of top interest will be a brief presentation by Frank E. Richart of his paper which appeared in two parts in the October and November JOURNALS—"Reinforced Concrete Wall and Column Footings." The information in this paper also indicates consideration should be given to possible changes in building code requirements for footings.

Luncheon—awards

At a **Thursday noon** luncheon ACI awards will be presented to: Frank H. Jackson and Harold Allen—Wason medals for the most meritorious paper; Richard C. Mielenz, Kenneth T. Greene and Elton J. Benton—Wason medals for noteworthy research; and Raymond E. Davis, E. Clinton Jansen and W. T. Neelands—Construction Practice Awards.

F. E. Richart

New officers

The **Thursday afternoon** session will start after a breather following the noon luncheon. Heading the program will be the report of the annual election with the

introduction and installation of officers. This year, according to new By-laws provisions, the officers will be installed immediately following the announcement of the canvass of ballots by the tellers. Retiring president R. F. Blanks will deliver the annual President's Address.

R. F. Blanks

Ready mixed concrete

Next, Stanton Walker, Director of Engineering, National Ready Mixed Concrete Association, will speak for about twenty minutes on "Control in Ready-Mixed Concrete Manufacture and Distribution." Mr. Walker's paper outlines major features in control of quality of concrete in the field, from the point of view of proportioning, mixing and transportation, with particular emphasis on those factors peculiar to the ready mixed concrete operation —central mixing and transit mixing. A primary purpose is to present points which may serve as a basis for floor discussion. Something is said about the mutual responsibilities of the ready mixed concrete producer and the contractor in maintaining quality. Lively and constructive discussion on the subject is expected from both users and manufacturers of ready-mixed concrete.

Portland cement paint application

There will follow a brief digest of ACI Committee 616 report (published in full in the January JOURNAL), "Proposed Recommended Practice for the Application of Portland Cement Paint to Concrete Surfaces." G. E. Burnett, chairman of the committee will present the report on a motion for adoption as an ACI Standard.

Ten research topics

Friday morning will be the annual open session of ACI Committee 115—Research, with chairman S. J. Chamberlin presiding. Ten or more reports will be given by research men on timely topics including, among others, the newest results from the extensive bond tests currently in progress. Reports are also expected on the investigation of prestressed spirally reinforced columns; use of electric strain gages inside concrete members; the effect of internal temperature variation, shrinkage and plastic flow in causing modification of stress distribution near the computed position of contraflexure in the interior spans of continuous reinforced concrete slab bridges; effect of restraint and durability characteristics on concrete aggregates; effect of rate of loading upon stress-strain curves of concrete beyond the ultimate strength; protective and decorative coatings for concrete masonry structures; some properties of lightweight aggregate concrete; a method for determining the air content of hardened concrete and some additional information on air entrainment in mass concrete.

Stanton Walker

The projects to be reported upon are not far enough along for publication; none of them is past the active stage and some are in only the preliminary stage. Some may be published in the near future; some perhaps not for years. Attendance at the session is the only sure way of being in on these previews of the many future developments in concrete. To permit the speakers and discussers from the floor to feel free to discuss possible trends and to raise questions without embarrassment or fear of being quoted, no transcript is made of the session.

Architectural concrete—lightweight concrete

The **Friday afternoon** session will also be one of great interest to architects, structural engineers and especially to construction superintendents and contractors.

J. J. Hogan

Two papers dealing with architectural concrete are of top interest—one on design details and one on construction practices by John J. Hogan and E. B. Oberly, respectively, of the Portland Cement Association's New York office. Ralph W. Kluge of Purdue University will describe, "Investigations of Lightweight Aggregate Concrete," undertaken while he was employed at the National Bureau of Standards.

Concrete masonry

Jay C. Ehle will report progress in concrete masonry unit manufacture.

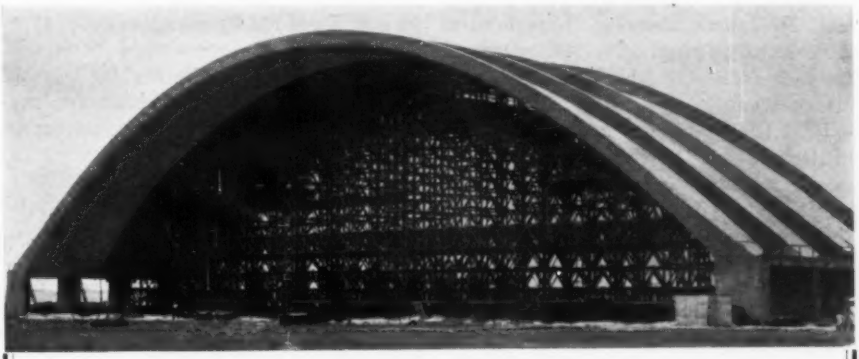
Make advance plans and reservations

As has been announced in previous information about the convention no evening sessions are planned because of the many opportunities for other activity.

For those who can spare the time, a few extra days in New York to visit construction projects of more than usual interest would be worthwhile. The Brooklyn-Queens Connecting Highway; the reinforced concrete frame and slab-band construction features of the Lillian Wald Houses and the New York Life Insurance Co. housing developments; New Jersey's new 8-lane highway from Elizabeth to Newark Airport; and the New York International Airport at Idlewild in Queens are suggested.

E. B. Oberly

The New York local committee, under the chairmanship of M. J. McMillan, has offered to make the necessary arrangements when they know more about the number interested and the projects preferred.



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Who's Who This Month

E. W. Scripture, Jr. and Frank J. Litwinowicz

E. W. Scripture, Jr. and Frank J. Litwinowicz describe tests on "Some Factors Affecting Air Entrainment" on p. 433. Dr. Scripture is Vice President in charge of Research, Master Builders Co., Cleveland and an ACI member since 1931. This paper is the third in the JOURNAL this year of which he has been co-author.

Mr. Litwinowicz is senior research assistant in charge of the physical testing section in the Master Builders Co. research laboratory where he has been associated with Dr. Scripture since January 1946. In addition to this work he is completing his college course in chemical engineering, interrupted when he entered the army.

R. E. Glover, Owen J. Olsen, and Carl N. Zangar

Messrs. R. E. Glover, Owen J. Olsen and Carl N. Zangar, describe "Experimental Aids in Structural Concrete Design" (p. 445) as used by the U. S. Bureau of Reclamation.

R. E. Glover is Senior Engineer, Bureau of Reclamation, Denver, where he specializes in mathematical problems. An ACI member since 1936 he is the author of three previous JOURNAL papers: "Flow of Heat in Dams", "Calculation of Temperature Distribution in a Succession of Lifts Due to Release of Chemical Heat," Nov.-Dec., 1937, and "Earthquake Stresses in Frame Structures," April, 1942.

Owen J. Olsen has been with the structural research section of the Bureau since 1944 working on model tests and analyses. For ten years he has studied many phases of testing and control of structural materials with particular emphasis on the development of test apparatus and equipment. He was one of the early workers in the Bureau's mass concrete research program inaugurated in connection with Hoover Dam. He is a graduate of Utah State College, receiving his bachelor's degree in civil engineering there in 1930.

Since May 1941 Carl N. Zangar has been Head of Photoelastic Laboratory of the U. S. Bureau of Reclamation. His work in the laboratory since 1935 has included investigations of stresses in concrete and earth dams and their foundations, stresses around galleries in dams and stresses in corbel buttresses. He started with the Bureau in 1934 as a junior engineer on preliminary designs and cost estimates of Grand Coulee Dam. Prior to that time he was junior engineer on preliminary designs and cost estimates for the Columbia Basin Commission of the State of Washington. He is a 1932 graduate of Washington State College with a master of science degree in civil engineering.

Robert F. Blanks and W. A. Cordon

Robert F. Blanks and W. A. Cordon report Bureau of Reclamation experiences in "Practices, Experience and Tests with Air-Entraining Agents in Making Durable Concrete," (p. 469) scheduled for presentation at the annual convention February 1949.

Robert F. Blanks, President of ACI, needs little introduction. Since 1932 he has contributed frequently to JOURNAL literature and has served on many committees and the Board of Direction. He has been with the Bureau of Reclamation since 1925 and is now Chief, Engineering and Geological Control and Research.

For the past 8 years W. A. Cordon has been with the research laboratories of the Bureau of Reclamation, Denver, Colorado. Previous experience includes three years as concrete engineer and chief earthwork inspector during the construction of Island Park Dam, Crosscut Canal and Diversion Dam, Idaho. For two and one-half years Mr. Cordon was construction inspector at Grand Coulee Dam, Washington. He is a 1935 graduate in civil engineering from

Utah State Agricultural College. A member of ACI since 1937 he is at present secretary of ACI Committee 613 and a member of Committee 214.

Gerald Milsom

Based on 20 years experience with concrete floor finishing, water-proofing and pneumatically-placed concrete Gerald Milsom discusses "Concrete Floor Finishing," p. 489. He is president of Metallicrete Floor Co., Ltd., Montreal, Canada, specializing in concrete floor finishing, principally in new industrial buildings. Previous associations were as manager of Sternson Structural Specialties, Ltd., director of Gunite and Waterproofing, Ltd. and president of Construction Gunite, Ltd. During five war years he served in the RCAF and was released with the rank of Wing Commander.

J. R. Shank

J. R. Shank, Assistant Director, Engineering Experiment Station, Ohio State University, Columbus, where since 1917 he has been investigating various factors affecting concrete, contributes "Plastic Flow of Concrete at High Overloads" on p. 493. His paper "The Mechanics of Plastic Flow of Concrete" won the Wason Research Medal in 1936. He is also the author of several papers on bond and characteristics of reinforced concrete.

Safety Awards

The National Safety Council announces that it has established a plan to recognize through public interest awards exceptional service in safety performed by press, radio, magazines, motion pictures and advertising and advertising organizations. The standards on which this award is based are necessarily high but the Council hopes that editors, radio station managers, advertisers and others will not be backward about nominating themselves for the award which is noncompetitive and applies to all public information media and every phase of safety—traffic, homes, schools, farm, industrial. Further information can be obtained from National Safety Council, 20 North Wacker Drive, Chicago 6, Ill.

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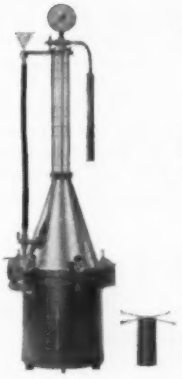

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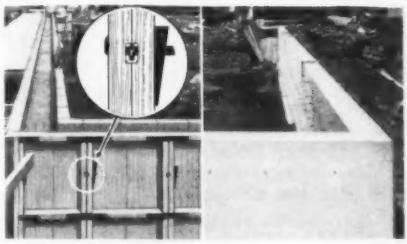

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New Members

Seventy-three applications for membership received in December were approved by the Board of Direction; of these 58 were Individual, 2 Corporation, 6 Junior, 7 Student.

New York was top membership state for the month providing ten new member applicants. California and Indiana were tied for second place with five applicants each.

The membership total January 1, 1949 was 4395.

Alabama

Millhouse, S. R., (Indiv.) Universal Atlas Cement Co., 630 Brown Marx Bldg., Birmingham, Ala.

California

Glaser, Samuel J., (Indiv.) 1001 N. Vermont Ave., Los Angeles 27, Calif.
Goethals, A. A., (Indiv.) 621 S. Hope St., Los Angeles 14, Calif.
Leavitt, George Edward, (Indiv.) c/o Pacific Coast Aggregates, Inc., Fair Oaks, Calif.
Sauer, Arthur A., (Indiv.) 2203 13th St., Sacramento, Calif.
Thomas, Ivor R., (Indiv.) 1911 San Rafael St., San Leandro, Calif.

Colorado

Benton, Elton J., (Indiv.) 3105 W. Ohio Ave., Denver 9, Colo.
Rao, J. Visweswara, (Indiv.) 1486 Clayton St., Denver, Colo.
Schlue, Edwin W., (St.) 2208 Arapahoe St., Golden, Colo.

District of Columbia

Clover, Paul B., (Indiv.) 1928 Eye St. NW, Washington 6, D. C.

Florida

Fritz, Irwin W., (Indiv.) 3517 Dubsdread Circle, Orlando, Fla.

Illinois

Grindel, John F., (Indiv.) 14436 S. LaSalle St., Chicago 27, Ill.
Haltiwanger, John David, (St.) 308 E. Healey St., Champaign, Ill.
Patrick, Joe Peeler (Indiv.) 1551 W. Olive Ave., Chicago 26, Ill.
Tachau, Herman, (Indiv.) 6823 S. Chappel Ave., Chicago 49, Ill.

Indiana

Higgs, Joe Garland, (Jr.) Joint Highway Research Project, Purdue University, W. Lafayette, Ind.
Ibanez G., Raul, (St.) 276 Dillon Hall, University of Notre Dame, Notre Dame Ind.
Leser, George N., (St.) 133 Howard Hall, Notre Dame, Ind.
Mauel, Edward, (Indiv.) 623 W. 39th Ave. Gary, Ind.
Scibelli, Louis Antonio, (St.) 756 Portage Ave., South Bend, Ind.

Iowa

Furlong, Wray C., (Indiv.) 802 Hubbell Bldg., Des Moines, Ia.
Wang, Ch'uan Chih, (Jr.) 211 S. Hyland, Ames, Ia.

Louisiana

Pryor, Harry W., (St.) 519 W. Mississsippi Ave., Ruston, La.

Maryland

Vollmer, A. R., (Indiv.) 1304 St. Paul St., Baltimore 2, Md.

Massachusetts

Crowell, John Thomas, (Jr.) 39 South St., Brockton, Mass.
Wachrameeff, Artemy A., (Indiv.) 157 Beacon St., Boston 16, Mass.

Michigan

Cash, C. H., (Indiv.) State Highway Dept., Lansing 13, Mich.
De Clerk, George J., (Indiv.) 6497 Ten Mile Rd., Centerline, Mich.

Mississippi

White, Walter Huntsville Jr., (Indiv.) 1299 N. President St., Jackson 23, Miss.

Missouri

Reineri, John, (Indiv.) Reineri Contracting Co., 1140 Ferguson Ave., University City, Mo.
Wilson, George B., (Indiv.) Room 300, Norwood Hall, University of Missouri, School of Mines & Metallurgy, Rolla, Mo.

New Jersey

Fuller, George A., Company, (Corp.) 14 Bayard Ave., Arlington, N. J. (Peter Dejongh)
Mahler, Martin, (Indiv.) 930 Burlington Ave., Union, N. J.

New Mexico

Lamborn, Ollie A., (Indiv.) 215 N. Mesa Verde Ave., Albuquerque, N. M.

New York

Blaha, Wm. A., (Indiv.) 1476 Broadway, New York, N. Y.

De Serio, James N., (Indiv.) 507 Root Bldg., Buffalo 2, N. Y.

Dohn, Charles W., (Indiv.) 70-41 67th Place, Glendale, L. I. 27, N. Y.

Francis, Frederick O., (Indiv.) 405 Sidway Bldg., Buffalo 3, N. Y.

Frewen, Robert J., (Indiv.) 34-51 83rd St., Jackson Heights, N. Y.

Harris, Elliott Jr., (Indiv.) 30 Stoneyside Dr., Larchmont, N. Y.

Kauffeld, Theodore J., (Indiv.) Devenco, Inc., 150 Broadway, New York 7, N. Y.

Soffer, George J., (St.) 380 E. 173rd St., Bronx 57, N. Y.

Tasio, Pascal L., (Indiv.) Master Builders Co., 101 Park Ave., New York 17, N.Y.

Zaborowski, Robert, (Indiv.) 119 Vanderbilt Ave., Manhasset, L. I., N. Y.

North Carolina

Dale, Frederick Amerman, (Indiv.) Cashiers, N. C.

Horton, Frank, (Indiv.) 1018 11th Ave., Hickory, N. C.

Ohio

Chamberlain, Joseph J. Jr., (Indiv.) University of Dayton, Dayton 9, Ohio

Huff, Marion S., (Indiv.) 6300 White Ave., Cleveland 3, Ohio

Vandegrif, Louis E., (Indiv.) 6889 Olentangy River Rd., Worthington, RFD, Ohio

Oregon

Burnham, Tom Eugene, (Indiv.) Eugene Sand & Gravel Co., P. O. Box 1067, Eugene, Ore.

Pennsylvania

Chapel, Karl F., (Indiv.) 42nd & Davison Sts., Pittsburgh 1, Pa.

Conaway, Don Russell, (Jr.) 2237 Spring Garden St., Philadelphia 30, Pa.

Lutz, Charles Earl, (Indiv.) 500 Brushton Ave., Pittsburgh 21, Pa.

Pressler, W. R., (Indiv.) 238 Cherry St., Philadelphia 6, Pa.

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Continued on p. 12

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Texas

Marquette, L. L., (Indiv.) 1511 14th St., Galena Park, Texas

Robinson, J. E., (Indiv.) P. O. Box 5981, Jasper, Texas

Washington

Rhodes, Fred H. Jr., (Indiv.) Civil Engineering Dept., University of Washington, Seattle 5, Wash.

Sharp, Lloyd Wellington, (Indiv.) U. S. Army, Corps of Engineers, Walla Walla, Wash.

Puerto Rico

Schimmelpfennig, W. H., (Indiv.) Watlington Apt. No. 6, Santurce, Puerto Rico

Brazil

Silva, Elato, (Indiv.) Rua Santa Adelaide, 235, (Perdizes) Sao Paulo, Brazil, S. A.

Canada

Dearborn, Fred R., (Jr.) 93 Prince William St., St. John, N. B., Canada

Ramsey, Thomas Alfred, (Indiv.) Central YMCA, Room 807, 1441 Drummond St., Montreal, Que., Canada

Dominican Republic

Bonnelly G., Rafael, (Indiv.) 40 Danae St., Ciudad, Trujillo, R. D.

England

Ozieblowski, Czeslaw, (Indiv.) M. I. Pol. E., York House, York Place, Port-Talbot, England

Malaya

Konstantinoff, B., (Indiv.) c/o United Engineers, Ltd., River Valley Rd., Singapore, Malaya

Mexico

Alvarez M., Carlos, (Indiv.) Lopez Cotilla 813, Colonia del Valle, Mexico D. F., Mexico

Dominguez-Lara, Renan G., (Jr.) Apartado 136, Merida, Yucatan, Mexico

South India

Rao, N. K. V. Subba, (Indiv.) Concrete Laboratory, P. W. D., Chepauk, Maddras, S. India

Southern Iran

Clapp, Leslie Rex, (Indiv.) c/o The Anglo Iranian Oil Co., Agha Jari, Via Abadan, Southern Iran

Switzerland

Burgdorfer, Ernest, (Indiv.) Gutenbergstr. 14, Berne, Switzerland

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Positions and Projects — ACI Members

Charles E. Wuerpel

Charles E. Wuerpel, active member of ACI since 1937, former chief of the Concrete Research Division, Corps of Engineers, U. S. Army, has been appointed technical director of Marquette Cement Manufacturing Co. He recently returned from Argentina where he was employed by the Inter-American Construction Corp. in Buenos Aires in connection with major construction work in Argentina. During the 19 years he was employed by the Corps of Engineers, he conducted a number of major research projects dealing with the development and use of concrete. In recent years he has published notable papers on air-entrainment in concrete, the work reported in one of them "Laboratory Studies of Concrete Containing Air-Entraining Admixtures" which was published in the ACI JOURNAL, Feb. 1946 won him the Wason medal for noteworthy research in 1946. An earlier work "Field Use of Cement Containing Vinsol Resin," ACI JOURNAL, Sept. 1945, and the laboratory study were combined with a symposium into ACI's latest book on air-entrainment in concrete. Long active in ACI affairs, Mr. Wuerpel is presently a member of Committee 115—Research, Committee 214—Evaluation of Results of Compression Tests in Field Concrete, Committee 604—Winter Concreting Methods, and is ACI representative on A.S.T.M Committee C-1, Cement.

Jack R. Benjamin

Jack R. Benjamin has been appointed full professor of engineering on the faculty of the Stanford University. He was formerly assistant professor of structural design at Renssalaer Polytechnic Institute and more recently assistant professor of civil engineering at Stanford.

Sigvald Johannesson

After 25 years service, Sigvald Johannesson, designer of the Pulaski Skyway in north New Jersey, has retired from the New Jersey State Highway Department. Among other achievements he developed the method of moving concrete pavements sideways with compressed air to widen the New Brunswick Pike.

Highway engineers honored

Four engineers were honored at the annual meeting of the Highway Research Board for their contributions to highway development. They are C. R. Hanes, field engineer, Ohio Department of Highways, who received the Board's annual award for the best paper presented at the 1947 meeting of HRB; Frank H. Jackson, principal engineer of tests, Public Roads Administration, and Charles H. Scholer, professor of applied mechanics, Kansas State College, recipient of the Board's new Distinguished Service Award; and G. Donald Kennedy, vice-president, Automotive Safety Foundation, recipient of the George S. Bartlett Award, an honor sponsored jointly by the Highway Research Board, the American Association of State Highway Officials and the American Road Builders Association. Messrs. Jackson and Scholer are members of the ACI Board of Direction.

C. A. Wilson reports on A.I.S.I. concrete research program

C. A. Wilson, research engineer of the Committee on Reinforced Concrete Research of the American Iron and Steel Institute, presented a report of that Committee to the 28th annual meeting of the Highway Research Board in Washington, December 10, 1948. In summarizing the work of the committee, Mr. Wilson mentioned the work of his predecessor, Roy Zipprodt, whose active participation in the work was ended by his untimely death in June 1947. He also described briefly the worthwhile research projects undertaken at leading educational institutions and at the National Bureau of Standards. Research projects have been completed or are

now under way at the University of Illinois under the direction of Professor Frank E. Richart—"Reinforced Concrete Wall and Column Footings," (published in the October and November 1948 issues of the JOURNAL); at Cornell University under the direction of Professors George Winter and Robert Mains—A Study of the Longitudinal Distribution of Bond Stress in Reinforced Concrete; at Iowa State College two small research projects under the direction of Professors H. J. Gilkey and S. J. Chamberlin, one relating to the spacing of bars in joists and beams and the other relating to lapped splices; at the University of Wisconsin, a research project devoted to the study of plastic flow on reinforced concrete slabs and beams under the direction of Professor George W. Washa. Work at the National Bureau of Standards on the bond of concrete to reinforcing steel was under the direction of Arthur P. Clark. Mr. Clark's report on this project was published in the December 1946 issue of the JOURNAL, "Comparative Bond Efficiency of Deformed Concrete Reinforcing Bars."

Robert J. Potts

Robert J. Potts of Harlingen, Texas, for many years professor of highway engineering at Texas A. & M. College, will become a member of the Texas Highway Commission on February 15, 1949. He is now serving his second year as president of the Texas Good Roads Association. After graduating from Texas A. & M. College, Mr. Potts taught highway engineering there and is the author of two books on road-building.

Lehigh University to conduct research on shell roof structures

A two year research project aimed at checking and improving design procedures of shell roof structures has been assigned to the Fritz Engineering Laboratory of Lehigh Univ. by the Roberts and Schaefer Co. of Chicago, according to an announcement by the University.

Under the contract, experimental and analytical research will be conducted on the design of shell structures of reinforced concrete of the type used in large airplane hangars or in domes of monumental buildings. In such structures spans of several hundred feet are bridged by shells only a few inches thick and stiffened by slender arch ribs.

The Roberts and Schaefer Co. has pioneered the application of theoretical analyses to the design of shell structures in this country and has engineered some of the largest airplane hangars in existence. The investigation now being initiated will seek experimental verification of existing theoretical procedures together with such modifications and improvements as may become apparent as the work proceeds.

Henry L. Bowlby

Henry L. Bowlby of Chicago, Ill., died on November 7, 1948 at the age of 68. A pioneer in road construction in the Pacific Northwest, Mr. Bowlby served as highway commissioner for the State of Washington, and as state highway engineer for the newly established Oregon State Highway Department. After service overseas in World War I, with the rank of lieutenant colonel, he became senior highway engineer for the U. S. Bureau of Public Roads (now the Public Roads Administration) and later was chief engineer of the Long Island State Park Commission. From 1932 to 1939 he was president of the National Concrete Service Corp., Chicago. He had been an ACI member since 1932.

Frank H. Mason

Frank H. Mason, Chief Engineer of the New England Public Service Company for over 40 years and long a member of the American Concrete Institute (1925) died after a short illness in Augusta, Maine on December 17th, 1948. In addition to ACI, Mr. Mason was a member of the ASCE, AIEE, the Boston Society of Civil Engineers and the Maine Association of Engineers.

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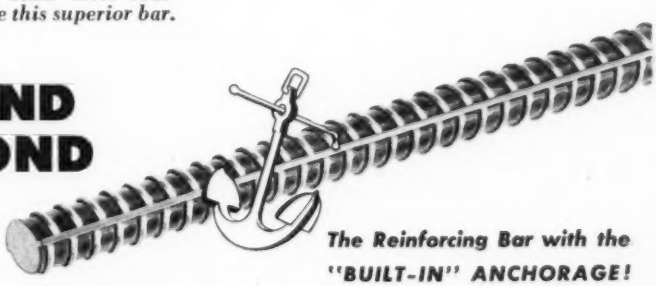

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Honor Roll

February 1, 1948 to December 31, 1948

The four top men in performance as Member getters for the previous eleven months are in the same relative positions on the following list as on December 1, 1938 (see January JOURNAL) and with unchanged Member credits. In fact Professors Morgan, Senior and Junior respectively, with their combined total of sixty-three new ACI Members sponsored, account for approximately ten percent of ACI's net gain in membership in 1948. With the two Californians, Davis and Stanton, in third and fourth positions, the only marked change is that Professor McCarthy, Indiana (these professors!) has crowded Howard Simpson, Massachusetts out of fifth place. With little time left in the Honor Roll year (ends January 31) changes at year's-end are still possible because as this goes to the printer the membership department is busy with an influx of new applications.

Newlin D. Morgan (Ill.).....33
Newlin D. Morgan, Jr. (Wyo.).....30
Raymond E. Davis (Calif.).....15
Thomas E. Stanton (Calif.).....14
James A. McCarthy (Ind.).....10½
Howard Simpson (Mass.).....10
Henry L. Kennedy (Mass.).....7½
Leon Venegas (Costa Rica).....7
Raphael Callejas H. (Guatemala).....6
Harrison F. Gonnerman (Ill.).....6
Oliver G. Julian (Mass.)..... 5
Wm. H. Thoman (Colo.).....4½
Harold S. Carter (Utah).....4
E. I. Fiesenheiser (Ill.).....4
Thomas C. Kavanagh (Pa.).....4
R. H. Sherlock (Mich.).....4
Lewis H. Tuthill (Colo.).....4
Jose Antonio Vila (Cuba).....4
Phil M. Ferguson (Texas).....3½
Elmo C. Higginson (Colo.).....3½
Myle J. Holley, Jr. (Mass.).....3½
A. E. Mackney (Canada).....3½
J. Antonio Thomen (Dominican Republic).....3½
Bailey Tremper (Wash.).....3½
G. E. Troxell (Calif.).....3½
Stanton Walker (D. C.).....3½
Robert W. Freeman (Calif.).....3
Ray C. Giddings (Calif.).....3
Edward L. Howard, Jr. (Calif.).....3
John C. King (Ohio).....3
George E. Large (Ohio).....3
F. N. Menefee (Mich.).....3
Robert B. B. Moorman (Mo.).....3
Rene S. Pulido y Morales (Cuba).....3
Robert Morris (Colo.).....3
James M. Polatty (Ga.).....3
Richard A. Roberts (Calif.).....3
Thomas C. Shedd (Ill.).....3
Joseph J. Waddell (Calif.).....3
Alberto Dovali Jaime (Mexico).....2½
R. R. Kaufman (Ohio).....2½
George L. Otterson (Va.).....2½
Frank E. Richart (Ill.).....2½
John A. Ruhling (Ill.).....2½
John C. Sprague (Ga.).....2½
H. D. Sullivan (Wash.).....2½
Calvin T. Watts (La.).....2½
Boyd G. Anderson (N. Y.).....2
Hugh Barnes (Calif.).....2
F. Campus (Belgium).....2
Jose Luis Copacete (Puerto Rico).....2
John K. Class (Pa.).....2
Moses D. Cohan (N. Y.).....2
W. S. Cottingham (Wis.).....2
Jacob J. Creskoff (Pa.).....2
Walter Dardel (Switzerland).....2
J. Fruchtbaum (N. Y.).....2
H. J. Gilkey (Iowa).....2
Ernst Gruenwald (N. Y.).....2
William E. Lumb (Canada).....2
F. T. Mavis (Pa.).....2
Eugene Mirabelle (Mass.).....2
Hugh E. Odor (Wash.).....2
Walter H. Price (Colo.).....2
Jerome M. Raphael (Colo.).....2
Manuel Ray Rivero (Cuba).....2
John W. Robison (Colo.).....2
Georges Ronai (Turkey).....2
C. H. Scholer (Kans.).....2
Edward W. Scripture, Jr. (Ohio).....2
E. Copeland Snelgrove (England).....2
J. Neil Thompson (Texas).....2
E. W. Thorson (Colo.).....2
Juan Agustin Valle (Argentina).....2
C. D. Williams (Fla.).....2
George C. Alden (Ore.).....1½
Harold W. Brewer (Colo.) 1½
R. V. Chate (England).....1½
A. R. Collins (England).....1½
R. Torres Colondres (Puerto Rico).....1½
Harmer E. Davis (Calif.).....1½
George C. Ernst (Nebr.).....1½
Milton Fromer (Calif.).....1½
Francis R. McCabe (Ore.).....1½
W. E. Parker (Canada).....1½
Dean Peabody, Jr. (Mass.).....1½
Clarence Rawhouser (Colo.).....1½
Anton Rydland (Calif.).....1½
Roy T. Sessums (La.).....1½
James M. Shivley (Ark.).....1½

Harry F. Thomson (Mo.).....1½
John Tucker, Jr. (D. C.).....1½
Harold E. Wessman (Wash.).....1½
Ray A. Young (Wash.).....1½
Jerome O. Ackerman.....1
J. B. Alexander.....1
W. H. Allison.....1
A. Amirikian.....1
Edwin C. Anderson.....1
J. A. Bakker.....1
S. Balasubranmanyam.....1
George E. Barnes.....1
Paul H. Barnes.....1
S. B. Barnes.....1
F. Barona.....1
Frank H. Beinhauer.....1
Charles J. Belz.....1
Rene L. Bertin.....1
Robert F. Blanks.....1
A. J. Boase.....1
E. K. Borchard.....1
H. L. Bowman.....1
William S. Bridge.....1
Carlos D. Bullock.....1
H. E. Burr.....1
John Butkus.....1
A. Carbone.....1
R. A. Caughey.....1
Robert D. Chellis.....1
C. O. Christy.....1
Miles N. Clair.....1
Rodger B. Collons.....1
R. E. Copeland.....1
Jack E. Counts.....1
M. H. Cutler.....1
J. Slater Davidson, Jr.....1
A. N. Dutta Choudhury.....1
Joseph L. Fabian.....1
T. J. Farish.....1
A. Finlayson.....1
T. H. Fleming.....1
R. E. Fowle.....1
Zorislav Franjetich.....1
Bengt Friberg.....1
Eric Fridh.....1
Frank M. Fucik.....1
Carmelo Galindez.....1
Anthony J. Giardina.....1
Hernando Gonzalez.....1
George E. Goodall.....1
R. R. Gore.....1
Charles J. Grady.....1
Emil A. Gramstorff.....1
Eugene Guillard.....1
Elliot A. Haller.....1
J. R. G. Hanlon.....1
George E. Hatch.....1
F. E. Henman.....1
Walter Hess.....1
Samuel Hobbs.....1
W. C. Huntington.....1

Continued on p. 18

The ROLL-A-METER

A DEVICE FOR ACCURATELY MEASURING ENTRAINED AIR IN CONCRETE

Now for the first time the rolling method may be used with a conveniently small size sample with ease and accuracy regardless of barometric pressure or porosity of aggregates.

The Roll-A-Meter is an outstanding value, having other important uses. Used as a Pycnometer, it provides an excellent method for determining specific gravities of cement, sand, gravel and admixtures, quickly obtaining moisture content of aggregates and making mortar tests.

Precision Instrument of Solid Bronze

Specifications

Net weight.....18 lbs.
Height.....22 in.
Outside diameter at center.....8 in.
Volume of base.....130 cu. in.

Now widely used by government agencies, Highway Departments and laboratories.

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Piers M. Williams....1
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Oscar Vega Arguelles
Manuel A. Arosemena
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A. S. Brock
Boyd S. Brooks
L. Rees Brooks
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R. A. Burmeister
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Jose Luis Casas
Thomas F. Chace
Wilbur H. Chamberlin
Richard de Charms
J. E. Christiansen
James H. H. Chun
H. W. Chutter
Emil W. Colli
F. Coppen
Rolland Cravens
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Tools, Materials, Services

Under this heading note will be made from time to time of producer literature of presumed technical interest (and available from its source for the asking) to ACI users of tools, equipment, materials, accessories and special services.

Universal testing machine

The Baldwin Magazine, V. 4, No. 4, Third and Fourth Quarter, 1948 contains a number of featured articles on products of the Baldwin Locomotive Works and its subsidiaries. Of particular interest is the description of the World's most powerful universal testing machine—a five million pound capacity tension-compression machine installed at the Philadelphia Naval Base. Also of interest is an article on Davis Dam on the lower Colorado River. Railroad enthusiasts will find a number of striking pictures of locomotives and trains, both old and modern.

Books on concrete

Concrete Publications Limited, 14 Dartmouth Street, London, S.W. 1, England has released a 32-page bulletin of its technical books on concrete. Contents of each book are very briefly outlined and prices are quoted, both British and United States currency. 35 books, 3 Journals and 1 Annual are listed.

Flat slab construction

The basis of design of the "Smooth Ceilings" System is briefly described in a recently issued four-page folder distributed by the Smooth Ceilings System, 802 Metropolitan Life Building, Minneapolis 1, Minnesota. Typical details and tabular design data are shown for flat beamless floor construction.

Symposium on mineral aggregates

A.S.T.M. has announced the availability of the "Symposium on Mineral Aggregates", 15 extensive papers, which was presented at its 1948 annual meeting in Detroit. The Symposium was sponsored by Committee C-9 on Concrete and Concrete Aggregates and Committee D-4 on Road and Paving Materials.

In his introduction K. B. Woods, Purdue University, symposium committee head, states "This symposium is devoted to presentation and discussion of a group of papers dealing with the distribution, characteristics and uses of mineral aggregates. The papers are written primarily for those groups of individuals interested in the characteristics and uses of mineral aggregates who normally are not intimately connected with the aspects of research and testing."

The 250-page book, in heavy paper cover, can be obtained from A.S.T.M. Headquarters, 1916 Race St., Philadelphia 3, Pa., at $3.00 per copy; cloth binding, $3.65.

Anchor bolt "gun"

A powder-actuated driver which, it is claimed, will "pin" a pipe hangar to a concrete ceiling or wall, steel I-beam or column, in a few seconds has been developed. According to the manufacturers, Mine Safety Appliances Co., Pittsburgh, the small portable tool embeds studs in steel or masonry by the discharge of a blank cartridge. Once embedded, the studs are claimed to have tremendous holding power—up to several thousand lb.

Two sizes of studs—⅜-in. and ¼-in.—in several varieties are available—externally or internally threaded or plain, headed types.

Operation of the five-pound tool is simple. The stud, with the cartridge attached to it, is inserted in the barrel. The operator then rotates a spring-loaded safety arm 180 deg. to move the firing pin into position for contacting the cartridge primer. Holding the barrel against the material to be penetrated, the tool is pushed forward sharply to discharge the stud.

The original powder-actuated driver, of which this tool is a smaller copy, was developed by Mine Safety Appliances Co. for repair and salvage work by the U. S. Navy during World War II.

Concrete on the farm

A new Portland Cement Association pamphlet entitled "Farm Concrete", is intended to encourage the construction of durable and efficient rural buildings by rural contractors. It describes ways in which concrete is being used and the techniques employed in various parts of the country. Available only in the United States and Canada.

Portable industrial 'seismograph'

A new tool which it is claimed will help engineers in their continuing battle against vibration—a portable "seismograph" for use in industry—has been developed by The Barry Corporation of Cambridge, Mass., and the Westinghouse Electric Corporation, Baltimore, Md.

The first application has been made in measuring the vibration of walls in a textile mill. Because of its compactness and easy portability, it is expected that the instrument may be used to analyze vibration problems on machinery as well.

The industrial "seismograph", so-called because it utilizes the same principle employed in instruments registering earthquake shock, is a combination of a special seismic pendulum mounting designed and manufactured by The Barry Corporation, and the Westinghouse "Vibrograph".

Get ALL the Advantages of Air-entraining with PROTEX

Resists Cracking Due to Freezing and Thawing

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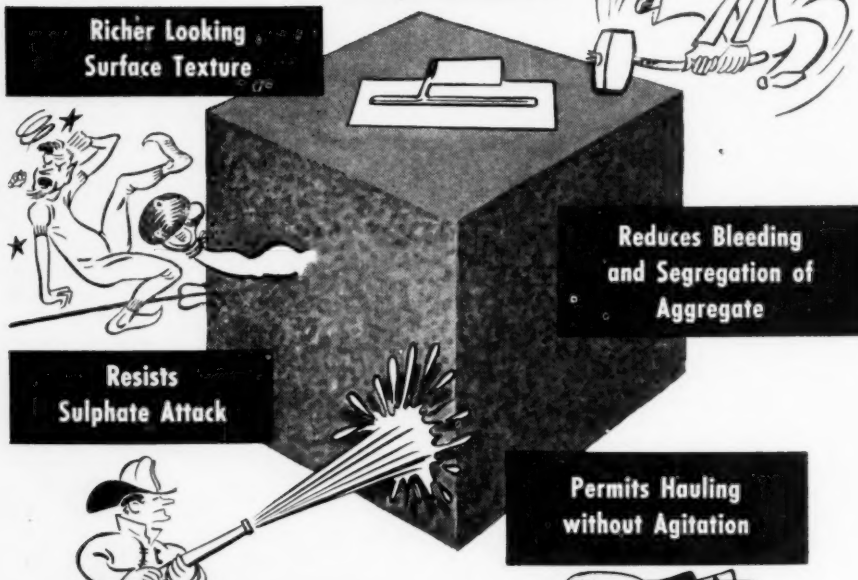

Reduces Bleeding and Segregation of Aggregate

Resists Sulphate Attack

Permits Hauling without Agitation

Greater Impermeability to Water

Protex . . . fully approved under ASTM C-175-48T and Federal spec. SS-C-192

FREE BOOK ON AIR ENTRAINMENT

FOR LOWER FINISHED COST PER YARD OF CONCRETE . . . USE **PROTEX AEA**

AUTOLENE LUBRICANTS CO.
Industrial & Research Division, Denver 9, Colorado
Please send me your book, "Facts on Modern Placement of Concrete Through Air Entrainment."
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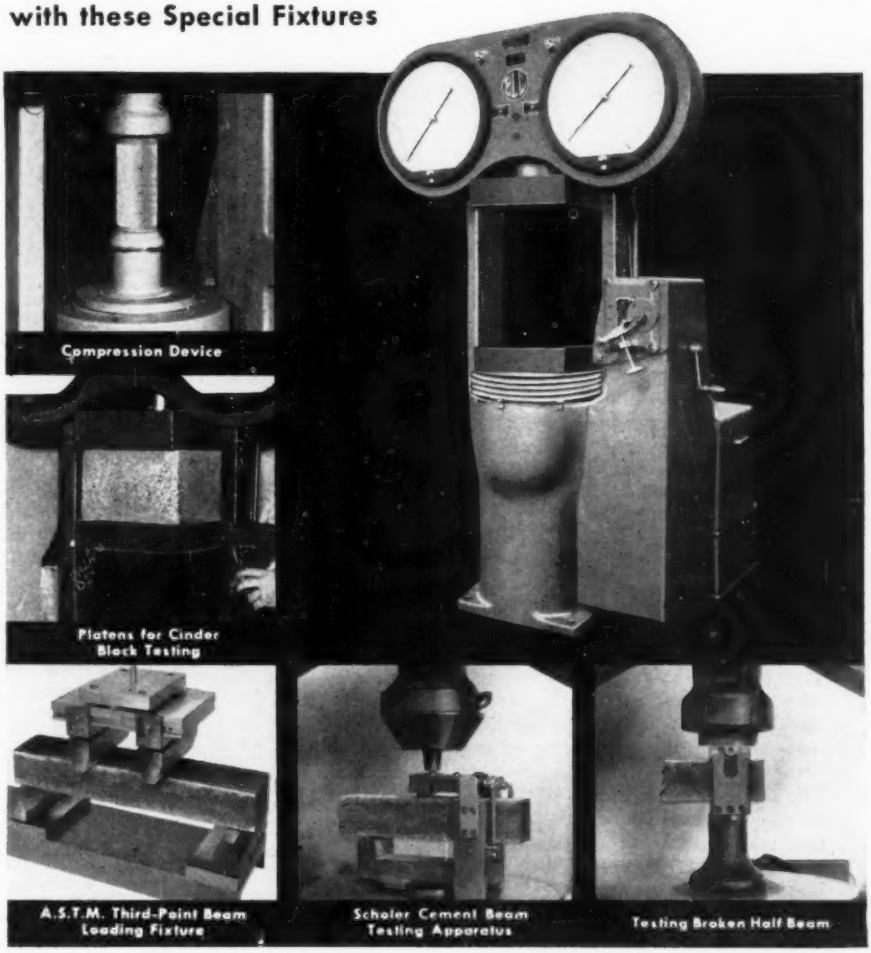
new fields of use for the **new**
BALDWIN Cement Testing Machine
with these Special Fixtures


Compression Device
Platens for Cinder Block Testing
A.S.T.M. Third-Point Beam Loading Fixture
Scholer Cement Beam Testing Apparatus
Testing Broken Half Beam

The broadened opportunities for cement and concrete testing opened by the new Baldwin 300,000 lb. machine are further expanded by available special fixtures.

These devices provide more detailed information, and facilitate the direct determination of such data as crushing strength, beam characteristics, and three-point loading reactions. Results provide valuable design factors which cannot always be inferred from straight compression tests.

The Baldwin 300,000 lb. testing machine meets all A.S.T.M. requirements, handles concrete cylinders 8″ x 16″ and building blocks up to 12″ wide x 18″ long. Write for details.

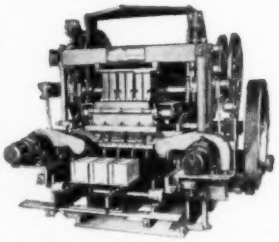
BALDWIN
TESTING HEADQUARTERS

The Baldwin Locomotive Works, Philadelphia 42, Pa., U. S. A. Offices: Birmingham, Boston, Chicago, Cleveland, Houston, New York, Philadelphia, Pittsburgh, San Francisco, Seattle, St. Louis, Washington. In Canada: Peacock Bros., Ltd., Montreal, Quebec.

BESSER VIBRAPAC BLOCK

FOR ALL TYPES OF STRUCTURES

To the BEAUTY and PERMANENCE of Besser Vibrapac Block must be added the VERSATILITY of this modern building material. Its many attractive and distinctive designs, as well as variety of sizes, can be applied equally well to any type of structure. Vibrapac control of block density and texture permit the manufacture of units required for insulation, soundproofing and acoustical control. The durability and design of Besser-made concrete masonry units stand as a building beacon to architects and builders throughout the world.



BESSER SUPER VIBRAPAC — the fully automatic, Plain Pallet block machine. Requires no machine operator. Pallets are fed automatically. Three 8″ x 8″ x 16″ block are made at a time on one Plain Pallet. Smaller sizes in larger multiples on the same pallets.

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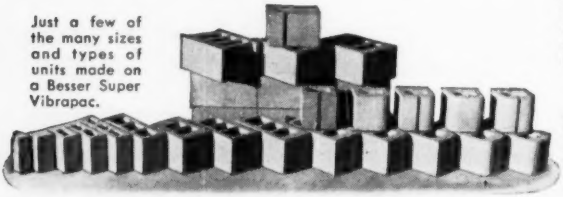
BESSER Vibrapac Better Concrete Masonry Units, in the Hands of Skilled Architects and Builders, Have Become in Effect a New Building Material.

Just a few of the many sizes and types of units made on a Besser Super Vibrapac.



Only ONE set of Plain Pallets required for ALL types and sizes of block. Speeds up block production. Reduces costs.

BESSER MANUFACTURING CO.

Complete Equipment for Concrete Products Plants

135-49TH STREET ALPENA, MICHIGAN, U. S. A.

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Just the right Vibrator...

Two CP-220 High-Frequency Electric Vibrators can be powered by a CP-2KW Gasoline-driven Generator

for every concrete job

Seven different models in the complete line of CP Vibrators enable the contractor to select just the type and size for a specific job.

— one man pneumatic and high frequency electric, small diameter models for reinforced concrete, walls and columns, etc.

— one-man and two-man pneumatic and high frequency vibrators for mass concrete.

Latest in the CP line is the light-weight, portable CP-220 high frequency vibrator for concretes of 3″ slump and over. Ask for Bulletin 814.

CHICAGO PNEUMATIC TOOL COMPANY

General Offices: 8 East 44th Street, New York 17, N. Y.

PNEUMATIC TOOLS • AIR COMPRESSORS • ELECTRIC TOOLS • DIESEL ENGINES
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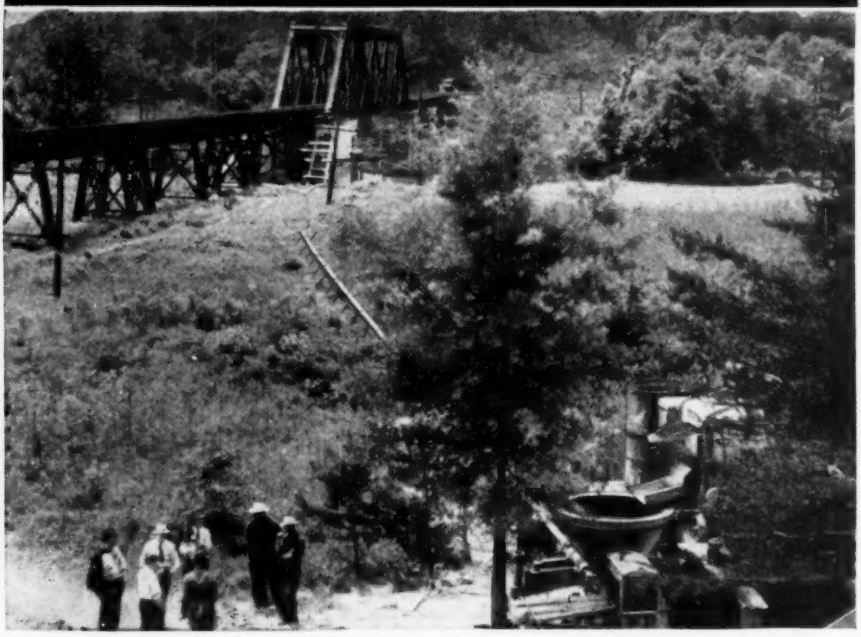


Quality with Economy

Rex Pumpcrete, the pump that pumps concrete through a pipeline, assures highest quality concrete because it reconditions the mix . . . transports it in a sealed pipeline, thus eliminating segregation. The concrete arrives at the end of the pipeline in the same high-quality condition as when it left the reconditioning hopper. "Concrete by pipeline" provides an efficient, economical method of concrete placement.

For example, on this job the Pumpcrete was used to place concrete around the old masonry of two main railroad bridge piers. The Pumpcrete was installed on an existing spur track *450 feet* from the construction site, and the concrete pumped to the piers. It's typical of the ability of Pumpcrete to place concrete without interfering with traffic or other operations.

On this job, as on so many others where quality concrete and fast, economical placement are required, the engineers recommended the Pumpcrete method. For facts on the Pumpcrete method, write Chain Belt Company, 1713 W. Bruce St., Milwaukee 4, Wis.

CHAIN BELT COMPANY REX CONSTRUCTION MACHINERY

"OUTSTANDING ROAD BUILDING UNITS"

'FLEX-PLANE' Joint Installing Machines for installing joints automatically or semi-automatically—we provide electric vibration so the concrete around the joint is thoroughly compacted to prevent cavities, and scaling.

'FLEX-PLANE' Spraying Machines—for curing concrete—an entirely automatic machine—each square yard of surface is double sprayed with proper amount of material.

'FLEX-PLANE' Traffic Line Marking Machines—for marking traffic lanes by use of pigment in plastic concrete.

'FLEX-PLANE' Dowel and Tie Bar Installing Machines —for installing dowels and tie bars in perfect alignment by vibration. No harness required.

'FLEX-PLANE' Finishing Machines—for finishing a complete surface in one pass with a compound screed—3115 lineal feet of 9″ concrete 24 feet wide in one 11 hour day—a world record!

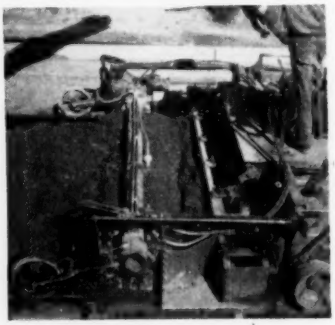
And Now Our Semi-Automatic Dowel and Tie-Bar Installer

FLEXIBLE ROAD JOINT MACHINE CO.

WARREN, OHIO, U. S. A.

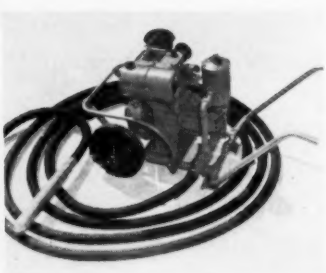
JACKSON VIBRATORS

For each and every purpose to which Vibrators are applicable in the concrete industry, we are confident we can supply the equipment that will give you not only the best and fastest placement, but also the maximum of dependability and trouble-free service. Pioneers and outstanding developers of vibratory equipment, our complete line includes internal and external vibrators for: General Construction * Light Construction * Mass Concrete Dam Construction * Hard-to-get-at Places * Form Vibrating * Floors, Streets and Highways * Pipe Manufacturing * Movement of Materials—Vibratory Tables—Thorough soil compaction of bridge approaches, similar fills and concrete floor sub-bases, etc. Drop us a line for complete information.



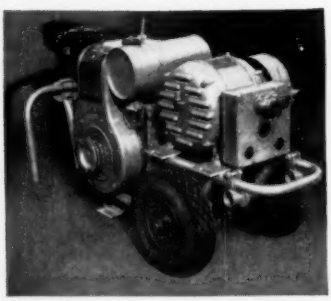
JACKSON HEAVY-DUTY VIBRATORY PAVING TUBE

For speedy full width highway and airport concrete paving. Up to 25 feet widths. Submergible dual tubes energized by powerful vibratory motors quickly transform harsh mixes to plastic state. Assures complete compaction — easy finishing. Cement savings up to 10% through reduction in W/C ratio. Attaches to any modern finisher, or rear of spreader for vibrating first course of extra thick slab construction. Variable frequency 3000-5000 V.P.M. Hydraulic lift. Grouped controls.



JACKSON HYDRAULIC CONCRETE VIBRATOR

This machine with its 50 ft. reach saves a great deal of time by making hard-to-get-to places quickly accessible and frequent relocations of the machine unnecessary. Operated by oil pumped through hose to hydraulic motor in vibrator head. Valve on power plant instantly adjusts frequency as desired, from idling to 7200 VPM. Vibrator head is 2¾ diameter. All moving parts run in oil. Hose may be reversed when wear finally occurs. Standard air-cooled gas engine, 5.7 HP. A very dependable, powerful and highly popular vibrator.



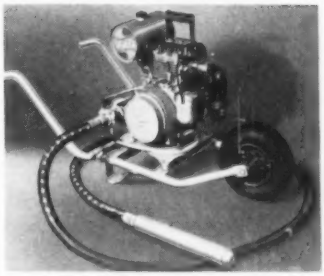
JACKSON PORTABLE POWER PLANTS

A new high in dependable portable power is now available in Jackson Power Plants having permanent magnet generators in which all usual maintenance is eliminated except lubrication. Especially designed for severest service under continuous operating conditions. Simple, rugged construction and design, stripped of fussy control gadgets. Run all Jackson Vibrators, for lights, and all types of contractor's power tools. One to 7.5 KVA sizes available.

ELECTRIC TAMPER & EQUIPMENT CO.,

The Line that Solves EVERY Concrete Vibrator Problem to BEST Advantage

JACKSON MODEL FS-6A FLEXIBLE SHAFT CONCRETE VIBRATOR



Compact and powerful (4 H.P. engine of national renown) built to take severe usage. Available with vibrator heads in 3 sizes and flexible sharting in 7′ and 14′ lengths up to 28′. Full-swivel, dirt-proof base. Furnished with or without wheelbarrow mounting. Frequency is instantly adjustable over a wide range. May be quickly converted to wet or dry grinding and drilling. An excellent vibrator for general construction work.

JACKSON Vibratory Soil Compactor

This remarkable new machine will give you up to 95% of maximum density (in granular soils) IN A SINGLE PASS—and do it in one-fifth the time the job may be attempted with other equipment. It is the ideal medium for "presettling" or firmly compacting bridge approaches, similar fills, and sub-bases for concrete floors which are to carry heavy machinery. Weighs only 150 lbs. Easily portable. Propels itself, under normal conditions. Firmly compacts 15 to 18 sq. ft. per minute (granular soil) to a depth of 12″. Time-tested vibratory motor—3-phase, 110 volt, 60 cycle A.C. Equally effective, with change of base, on coarse gravel, rock and for smoothing and compacting blacktop. 4 of these machines may be operated simultaneously by one Jackson (2.5 KVA) power plant.

JACKSON Municipal Paving Unit

Consists of an electric vibratory screed operated by a Jackson Power Plant of 1.25 KVA capacity. Screed is the only hand screed that will undercut at side forms, roll back for second passes, strike off crowns, both regular and inverted, permit operators to work from front, rear or sides. Has such a strong tendency to propel itself in the forward direction that only small effort is necessary to strike off stiff mixes. Manholes and storm sewers are no handicap due to the fact that screed does not reciprocate. Does an excellent job of vibrating concrete to depth of 10 inches on any slab from 6 feet on up to any practicable width. Radically reduces cost and greatly increases production.

LUDINGTON, MICHIGAN

HOW TO CONVEY BULK PORTLAND CEMENT ECONOMICALLY AND EFFICIENTLY

Bulletin FK-20 contains 24 pages describing and illustrating Fuller-Kinyon Conveying Systems for the handling of bulk cement. It shows how they are applied in ready-mix concrete and concrete products plants; how contractors use them on road building, lock and dam construction.

If you are interested in clean, economical and efficient handling of bulk Portland cement, write for Bulletin FK-20, Bulk Cement Conveying with Fuller-Kinyon Systems.

Whenever you have occasion to convey dry pulverized materials, get in touch with us. We will be glad to furnish estimates and layouts without any obligation.

P-94

FULLER COMPANY, CATASAUQUA, PA.

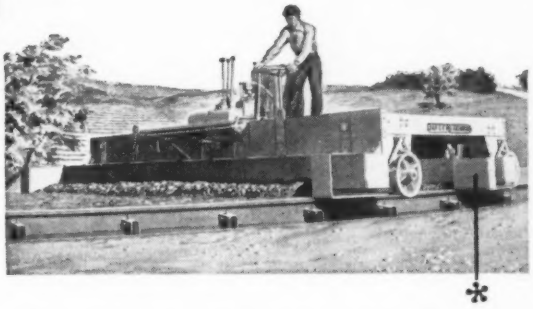
Chicago 3 - 120 So. LaSalle St.

San Francisco 4 - 420 Chancery Bldg.

A Labor-Saving, Mass-Production Paving Team

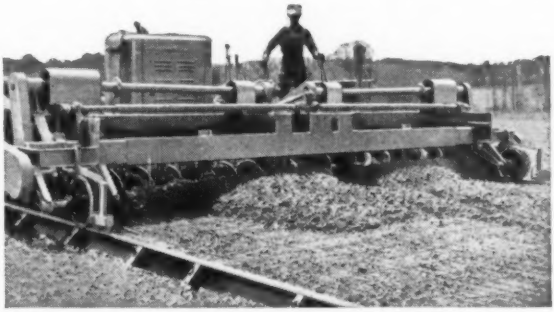
Jaeger engineered to match the pace of big, dual drum pavers.

TYPE "X" DIAGONAL SCREED FINISHER



*

Finishes flush to upper form on pitched slab and elevated curves, because its diagonally-adjustable rear screed* moves material uphill and compacts it solidly against upper form — greatly reduces carryback, saves 1 or 2 shovelers. Finishes far more accurately than transverse finishers, because diagonal screed* meets material at different points than transverse front screed — slices off high spots, fills in low ones. Finishes stiff mixtures fast and without tearing, saves extra passes. Fast and flexible — three working speeds forward and reverse, plus high travel speed. Three screed speeds — independent of traction speeds. Screeds have "velvet-touch" spring-cushioned finishing action, hydraulic lift, and single-lever quick crown change. Standard widths are 10′ to 15′ and 20′ to 25′ with 5-foot adjustability by 6″ increments. Special widths — 15′ to 20′ and 25′ to 32′.



and —

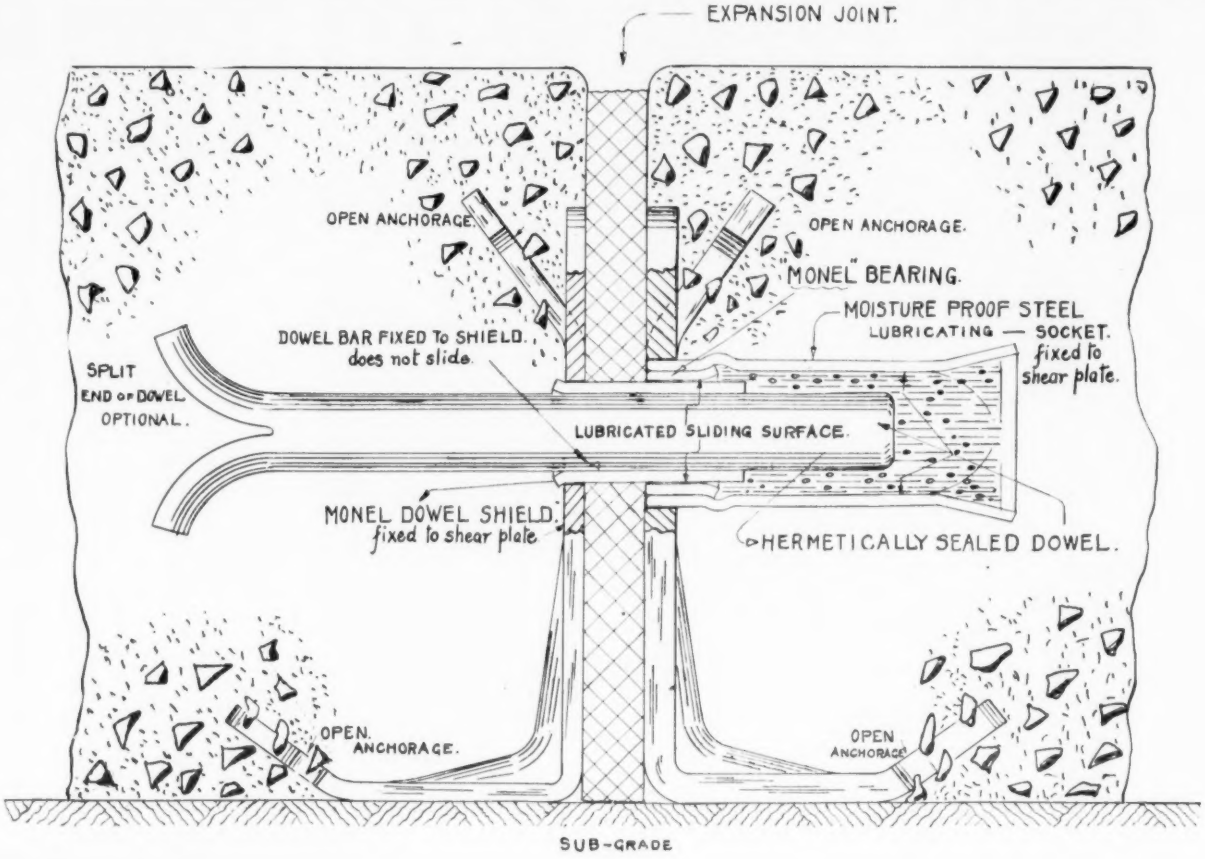
JAEGER RE-MIXING, COMPACTING SPREADER

A one-man machine that eliminates hand labor in the pit and leaves only precision work for the finisher. Will spread all the yardage which two 34E dual drum pavers can pour. Remixing-compacting spreading screw (exclusive with Jaeger) prevents segregation and honeycombing, produces dense uniform pavement — works material against subgrade and side forms, and eliminates variation between batches. Working range is from 3½″ below the forms to 3″ above them. The 10′ to 15′ model has vacuum controlled reversible screw — 20′ to 25′ model features dual reversible screws. Special widths from 15′ to 20′ and 25′ to 32′. Wide oscillating strike-off can be furnished for concrete work and bituminous spreading-finishing.

THE JAEGER MACHINE COMPANY, Columbus 16, Ohio

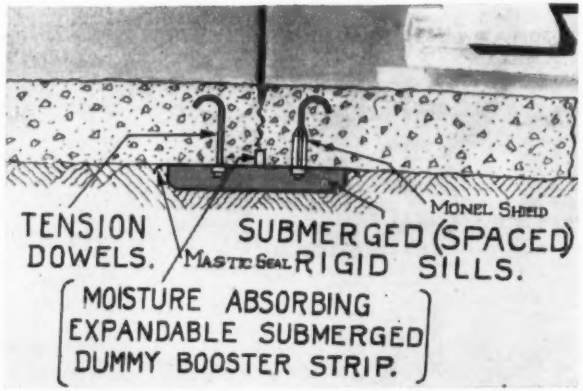
REGIONAL OFFICES: 1504 Widener Bldg. Philadelphia 7, Pa. · 226 N. La Salle St. Chicago 1, Ill. · 235 American Life Bldg. Birmingham 1, Ala.

Air Compressors • Pumps • Mixers • Truck Mixers • Hoists
Concrete Finishers • Bituminous Pavers



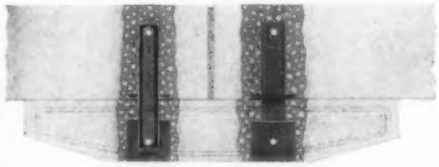
Scientists never

Scientists never gave up their determination to produce an Atomic Bomb.



Engineers never gave up their determination to secure a practical load transfer element for expansion and contraction joints for modern concrete highways and airports.

Heltzel engineers have the answer to every single expansion joint problem involving load transfer elements.



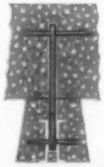
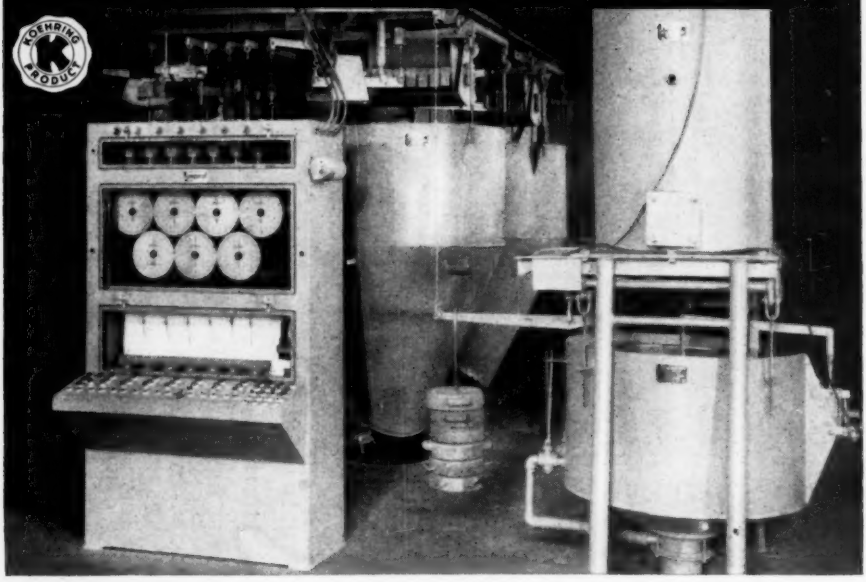


Fig. 4—Helco Tension Dowel and Expansion Joint Beam for Highways

Are you interested in permanently lubricated dowels, frictionless dowels—submerged load transfer elements, non-corrosive dowels, hermetically sealed dowels or tension dowels.

HELTZEL STEEL FORM & IRON CO.
WARREN, OHIO, U.S.A.

NERVE CENTER for RECORD PRODUCTION



JOHNSON Automatic Mixing Plants Offer YOU...

HIGH YARDAGE OUTPUT because...

Concentric zoning groups 1 to 6 mixers around center for most direct material flow, eliminates segregation of aggregates, and allows steep bottom slopes in aggregate hoppers for fast charging time. Central location of Cement Compartment permits steep slopes in cement tank for fast flow and maximum premixing action by introduction of cement into center of aggregate stream. Automatic Batcher Control features 2-step filling for fast and accurate weighing, instantaneous mix selection and fast automatic moisture compensation.

QUALITY CONTROL because...

Separate single-material Batcher units operated from a master control board (see photo above) are used for cement, water and each size aggregate. 2-step fill insures accurate weighing because intermittent feeding at close of charging operation reduces possibility of overload. Moisture compensator automatically corrects Water Batch scale to compensate for water content of sand. Adjustable retard on each Batcher gives optional delay in introducing Batch into mix.

ACCURATE PRODUCTION RECORDS because...

Chart pens operate directly from scale mechanism, record weight of each material on same sheet. Mixer consistency can also be shown. Electric counter records Batches discharged. Recorder also prints bars on chart paper, saving money and assuring accuracy of register.

THE C. S. JOHNSON COMPANY

Koehring Subsidiary Champaign, Illinois

KELLEY COMPACTOR POWER FLOAT

DELIVERS DURABLE, WEARING

FLOORS — DRIVEWAYS — PAVEMENTS — ROADS

CREATES A SAVING IN LABOR AND CEMENT COSTS

The Kelley Compactor Power Float at work on low water-cement ratio mix thirty minutes after placing. The weight of the machine, plus the action of the compactor hammers and the rotating of the disc, compacts and floats out the mix to a smooth, level surface, bringing just enough mortar to the surface for steel troweling.

The compactor hammer blows on the rotating disc as it floats, compacts and densifies the mix (drive the material in at the bond line where topping is applied to base) and deliver a dense, wear-resisting floor—A dustless floor—A floor on which the maintenance is reduced to the minimum.

ELECTRIC AND GASOLINE MODELS

Refer report Committee 804, "Concrete Wearing Surfaces for Floors," Sept. 1938 ACI Journal.

Booklet "Concrete Facts about Concrete Floor Finishes" sent on request.

KELLEY ELECTRIC MACHINE COMPANY

287 HINMAN AVE. (Zone 17) BUFFALO, N. Y.

TECHNICAL PAPERS GIVE RESULTS OF CONCRETE RESEARCH

Information we have developed in our research in concrete, particularly concerning cement dispersion and air entrainment, is given in the following papers, published and unpublished. Copies of these papers will be gladly supplied upon request.

Cement Dispersion

1. "Application of the Principle of Dispersion to Portland Cement."
2. "Economics of Cement Dispersion."
3. "Relation of Dispersion to Special Cements."
4. "Cement Dispersion and Admixtures."
5. "National Bureau of Standards Interim Reports Nos. 1, 2 & 3 Tests of Pozzolith for U.S.E.D."
6. "Portland Cement Dispersion by Adsorption of Calcium Lignosulphonate." Industrial and Engineering Chemistry Vol. 37 — June '45 Ernsberger & France.
7. "Some Physical and Chemical Properties of Weight Fractionated Lignosulphonic Acid, including the Dissociation of Lignosulphonates" Journal of Physical and Colloid Chemistry Vol. 52 — No. 1 — Jan. '48 — Ernsberger & France.
8. "Methods for Measuring the Passage of Water Through Concrete." A.S.T.M. Vol. 39 '39 — W. M. Dunagan.
9. "Portland Cement Dispersion . . . with Pozzolith."
10. "Pozzolith, Cement Dispersion, And The Water-Cement Ratio Law."

Air Entrainment

11. "Cement Dispersion and Air Entrainment in Concrete Pavement Construction."
12. "Influence of Size Grading of Sand on Air Entrainment." J.A.C.I. Nov. '48 — Vol. 20 No. 3— E. W. Scripture, Jr., F. B. Hornibrook & D. E. Bryant.
13. "Some Factors Affecting Air Entrainment." J.A.C.I. — Feb. '49 — E. W. Scripture, Jr. & F. Litwinowicz.
14. "Effect of Mixing Time on Air Entrainment." } E. W. Scripture, Jr. & F. Litwinowicz
15. "Effect of Batch Size on Air Entrainment." } (Unpublished — Mimeographed)
16. "Air-Entrainment with HP-7."

Heat Liberation

17. "The Effect of Various Reagents on the Heat Liberation Characteristics of Portland Cement," J.A.C.I. Vol. 37 Nov. '40 — L. R. Forbrich.
18. "Temperature Effects Near Concrete Surfaces as Affected by Heat Liberation of Cement." J.A.C.I. Vol. 13 Sept. '41 — L. R. Forbrich.

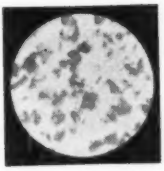
Floors

19. "Cement Dispersion and Concrete Floors."
20. "Metallic Aggregate in Concrete Floors." J.A.C.I. Sept.— Oct. '36 — Vol. 33 — E. W. Scripture, Jr.
21. "Some Tests of Corrosion Resistance in Concrete Floors with and without Metallic Aggregate." J.A.C.I. — Dec. '48 — Vol. 20 No. 4 — E. W. Scripture, Jr. & C. Sakryd.

Miscellaneous

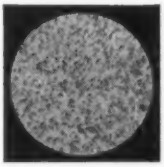
22. "The Action of Embeco In Concrete and Mortars." (Metallic Aggregate to control shrinkage.)
23. "Shrinkage of Mortar." Architectural Record — Nov. '35 — E. W. Scripture, Jr.

CEMENT DISPERSING ACTION OF POZZOLITH



FLOCCULATED-

PHOTOMICROGRAPHS OF CEMENT SUSPENSIONS



DISPERSED

(Left) Showing how cement particles clump together or flocculate when placed in water.

(Right) Pozzolith, through its component calcium lignosulfonate, causes the particles to separate by imparting to them like electrostatic charges. Dispersion of the cement produces these important effects: the water which had been trapped within the particle clumps is released to become a part of the mixing or placing water; the surface area in contact with the water is greatly increased since the particles are no longer in contact with each other. A certain amount of additional air is entrained. By these means the cement is more effectively used to produce concrete of improved properties.

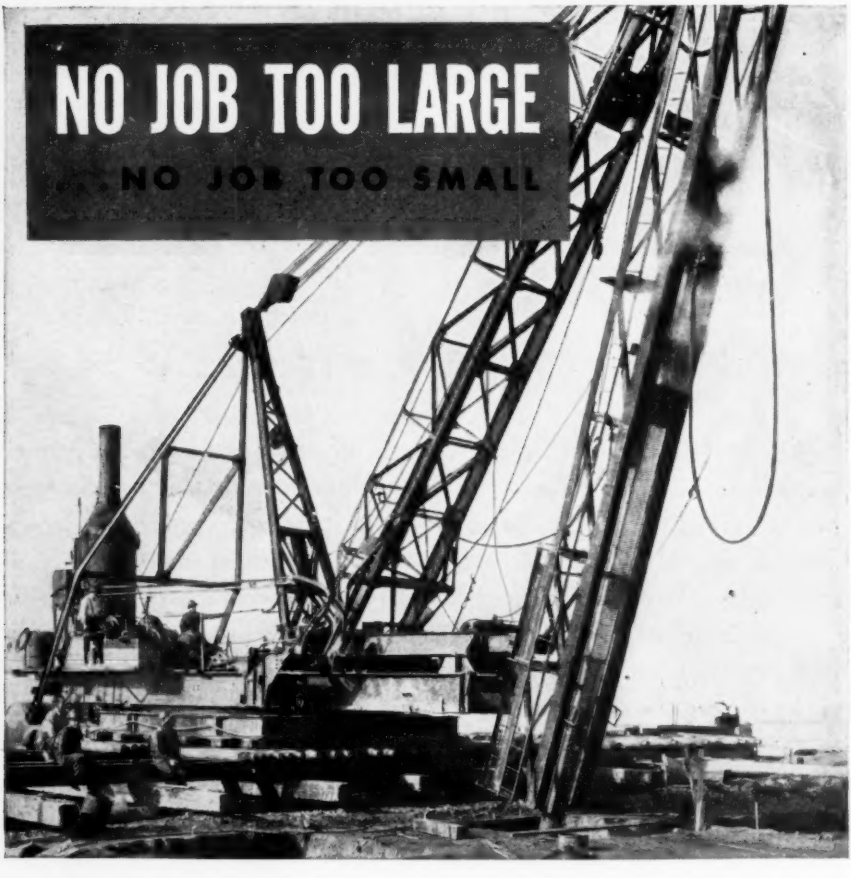
POZZOLITH COMPLIES WITH WATER-CEMENT RATIO LAW

Pozzolith, the cement dispersing and water-reducing admixture increases the efficiency of all cements 15% or more. When used with the ACI Standard Recommended Practice for the Design of Concrete Mixes (ACI 613-44) it produces the required strength, workability, watertightness and air content (durability), *most economically because it complies with the water-cement ratio law*. Findings of top testing authorities prove this. Pozzolith's large scale use denotes its wide acceptance and satisfactory field performance. (Pozzolith Concrete is now supplied by more than 300 leading ready-mix concrete producers.)

THE MASTER BUILDERS COMPANY

Department R

CLEVELAND 3, OHIO **TORONTO, ONTARIO**



Owners, architects, engineers and general contractors know that buildings rest safely on Raymond concrete pile foundations. That's why for more than half a century on projects around the world Raymond concrete piles have been used again and again—to provide safe subsurface support that is dependably permanent. Your inquiries receive prompt attention.

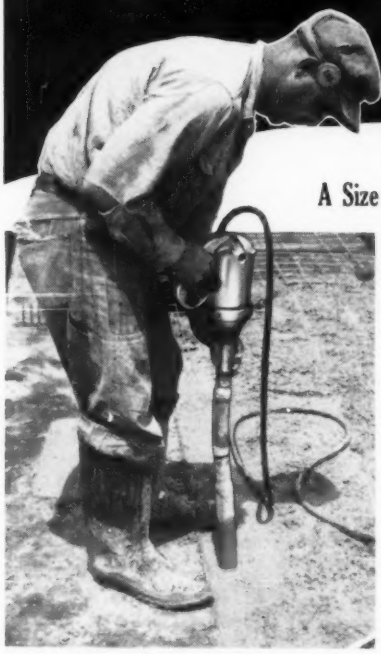
THE SCOPE OF RAYMOND'S ACTIVITIES includes every recognized type of foundation construction—concrete, composite, precast, steel, pipe and wood piles. Also caissons, underpinning, construction involving shore protection, shipbuilding facilities, harbor and river improvements and borings for soil investigation.

RAYMOND CONCRETE PILE CO.

Branch Offices in Principal Cities of United States and Latin America

140 CEDAR STREET, NEW YORK 6, N. Y.

Mall® VIBRATORS BRING OUT THE BEST IN CONCRETE



A Size and Model for Every Job

All of the economies and advantages you expect from concrete can be easily and quickly accomplished by vibration with Mall Vibrators. They place stiffer mixes than can be placed by hand, saving water, cement and sand, and permitting earlier removal of forms. They place concrete faster and better with less effort in areas inaccessible to hand methods. Honeycombs and voids are eliminated, saving costly hand patching. A better bond is achieved with reinforcement and successive layers, absorption is reduced, while a greater density, watertightness, increased compressive and flexural strength is secured. For their size, Mall Vibrators place more concrete than any other vibrator.

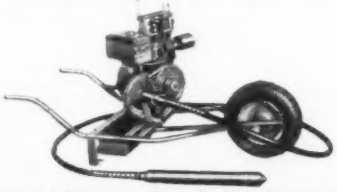
Mall One-Man Short-Coupled Electric Vibrator Delivers 7000 Frequencies per minute.

The new Mall One-Man Short-Coupled 1½ H.P. Electric Vibrator is a perfect tool for placing concrete in small forms. Also for edge vibrating and expansion joints on pavement work. The Mall 1½ H.P. and 3 H.P. Variable Speed Gasoline Engine Vibrators also operate tools for *Wet Wall Rubbing, Form Sanding, Wire Brushing, Pumping, Drilling, Sawing and Sharpening Tools.*

Mall 1½ H.P. Gasoline Engine Vibrator Delivers 4000 Frequencies per minute.

Write TODAY for complete literature.

Mall 1½ H.P. Universal Electric Vibrator Delivers 9000 Frequencies per minute. Also 7500 f.p.m. Pneumatic Unit.



Mall 3 H.P. Gasoline Engine Vibrator Delivers 3000 to 7000 Frequencies per min. Also available with round base.

MALL TOOL COMPANY

7703 South Chicago Avenue
CHICAGO 19, ILL.

"We Like Their High Discharge Speed And Low Maintenance Costs!"

D. B. CLARKE

The Fenton Fleet of Ransome Blue Brute Hi-Up Truck Mixers

Mr. D. B. Clarke, V. P. and Sales Manager of the H. G. Fenton Material Company, San Diego, Cal., writes in as follows:

"We purchased our first Ransome Hi-Up machine two years ago and have since added eight more. We have been well pleased with the performance of these mixers both from a standpoint of producing quality concrete and low maintenance costs, but particularly impressed with the speed that they discharge concrete with a slump as low as 1½ inches."

Note Mr. Clarke's last sentence about fast discharge with practically no slump. It means no time lost at the job, more hauls per day, and steady cost-cutting throughout the long life of every Ransome Blue Brute Hi-Up Truck Mixer.

In fact, speed is the keynote of Hi-Up performance—time- and money- saving speed based on the lasting efficiency of Blue Brute advanced design and construction. The non-jamming sealing door, quick-charging hopper and Ransome's famous mixing action shorten the time of every trip — while the trouble-free water system, easy accessibility of parts and strain-absorbing flexibility of design mean lowest possible maintenance costs and down-time.

You've read how H. G. Fenton Company's original purchase of two Ransome Hi-Ups was soon followed by expansion into a Hi-Up fleet. That's an old familiar story among Hi-Up users — and it will pay you to find out why. Get further proof that *there's more worth in a Blue Brute* from your Worthington-Ransome Distributor. Or write for Bulletin 221.

WORTHINGTON PUMP AND MACHINERY CORPORATION

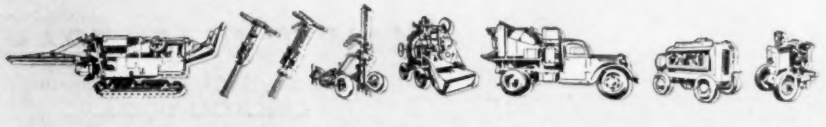
Construction Equipment Department, Harrison, New Jersey

Distributors in all principal cities

It's a battery of cost-cutters! Nine Blue Brute Hi-Up Truck Mixers owned by the H. G. Fenton Material Company, lined up at the firm's batching plant in San Diego, Cal. Machines are serviced by a Ransome 84S (3 yd.) Big Mixer. Fleet purchased through the Golden State Equipment Company, Los Angeles.

WORTHINGTON

BUY BLUE BRUTES



IF IT'S A CONSTRUCTION JOB, IT'S A BLUE BRUTE JOB

TECHKOTE PRODUCTS FOR CONCRETE

These products are the result of extensive research and development work. They have proved themselves on construction projects, large and small, throughout the West.

"SEALTEX" CONCRETE CURING COMPOUNDS

CLEAR — A clear curing membrane. It is a uniform solution with no tendency toward separation of ingredients. It contains a fading dye which marks the areas treated, thus avoiding excess coverage and waste. The dye fades out in a few days, leaving the concrete in its natural color.

WHITE-PIGMENTED — Developed for use where concrete may be subjected to elevated temperatures due to heat absorption from the rays of the sun. The white pigment substantially reduces this heat absorption. The resulting lower concrete temperatures during the early hardening period are beneficial because thermal stresses are reduced to a minimum.

GRAY-PIGMENTED — Essentially the same as White-Pigmented except that the complete white has been changed to a light gray cast. This prevents glare while retaining the high degree of heat reflectivity. The Gray-Pigmented is used primarily for streets, highways, and runways.

BLACK — A low cost, efficient curing compound. Developed for use where appearance of finished concrete work and heat absorption are not important factors.

These products meet the exacting specification requirements of the U. S. Bureau of Reclamation, U. S. Army, U. S. Navy, other Federal Agencies; State Highway Departments, Counties, Cities; Engineers and Architects.

TECHKOTE MASTIC JOINT SEALING COMPOUND COLD APPLIED, INTERNAL SET UP, WATERTIGHT

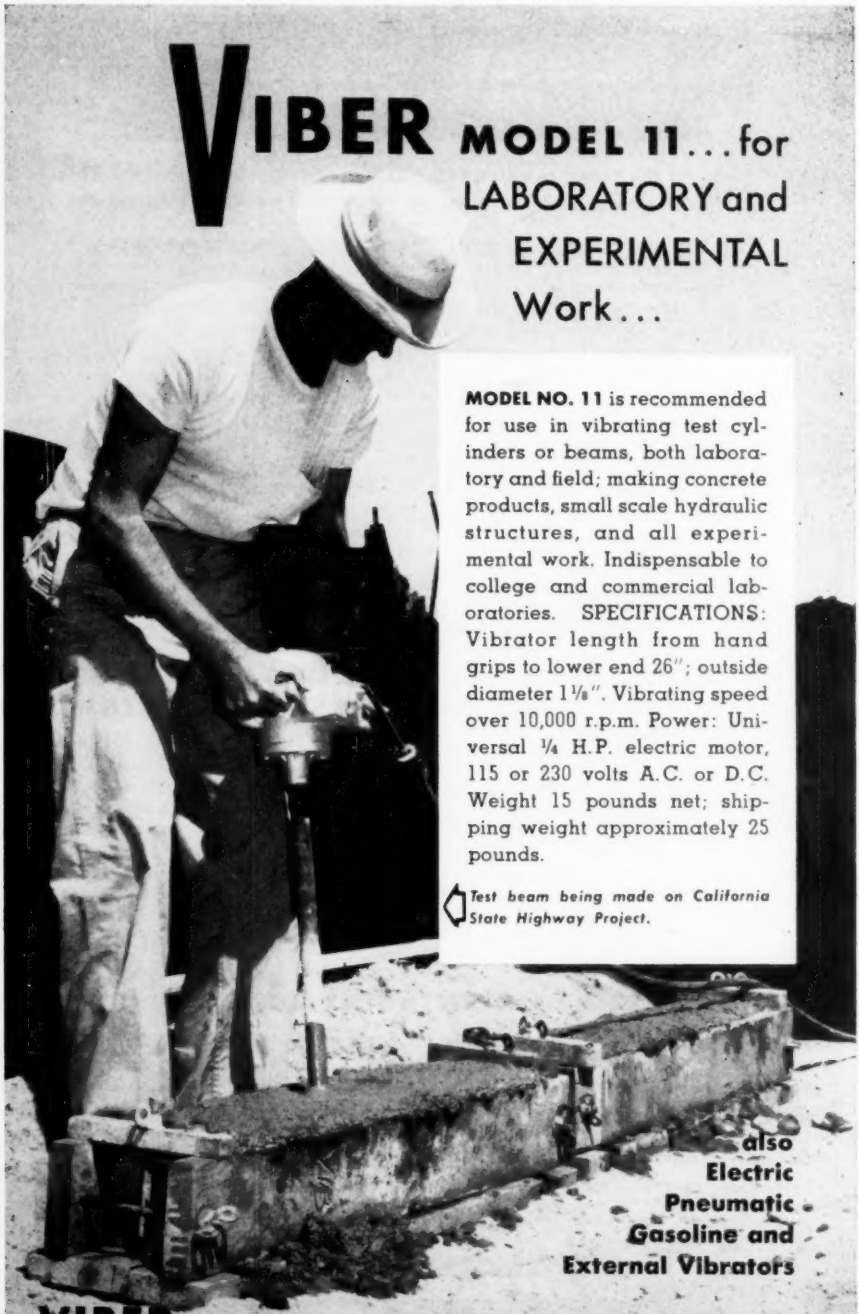
Advantages and characteristics are: (1) It is applied cold; (2) The initial consistency of the mix is such as to allow easy placement by troweling, by calking gun, or by air pressure gun; (3) It can be applied to the "green" concrete and will form a permanent bond; (4) It is specially designed for easy and efficient application to both vertical and inclined joints; (5) There is no shrinkage or flow of joint at any time; (6) After internal setting of the joint (approximately twenty-four hours), it forms a rubber-like material which will not crack or break the bond when extended 50% of its width at 0° F.; nor will it flow during hot weather. It is a permanent joint, passing the exacting specifications of the U. S. Bureau of Reclamation as well as the main requirements of Federal Spec. SS-F-336a and C.A.A. Spec. P-605; (7) Techkote Sealing Compound meets the most difficult job requirements and saves in time and labor.

"DURAIR" AIR-ENTRAINING AGENT

1. DURAIR is a highly active air-entraining agent for concrete.
2. DURAIR substantially increases the durability of concrete. Scaling is also greatly reduced.
3. DURAIR saves time and labor costs in mixing, placing, and finishing operations.
4. DURAIR results in better, more uniform concrete with less cement, at the same time affording a wider choice in the use of aggregate and sand.
5. DURAIR will not settle out in storage and will not gum up dispensing equipment. It can be used with any kind of cement and also in the presence of Calcium Chloride.
6. DURAIR'S small cost is more than compensated in the saving on materials and labor.

The TECHKOTE COMPANY
Inglewood, California

VIBER MODEL 11... for LABORATORY and EXPERIMENTAL Work...



MODEL NO. 11 is recommended for use in vibrating test cylinders or beams, both laboratory and field; making concrete products, small scale hydraulic structures, and all experimental work. Indispensable to college and commercial laboratories. SPECIFICATIONS: Vibrator length from hand grips to lower end 26"; outside diameter 1⅛". Vibrating speed over 10,000 r.p.m. Power: Universal ¼ H.P. electric motor, 115 or 230 volts A.C. or D.C. Weight 15 pounds net; shipping weight approximately 25 pounds.

Test beam being made on California State Highway Project.

also Electric Pneumatic Gasoline and External Vibrators

VIBER COMPANY • Internal Vibrators Since 1931

DEPT. 22, 726 SOUTH FLOWER ST. • BURBANK, CALIFORNIA

THIS IS HARD

BUT THIS IS EASY

AND THIS IS
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SYNOPSES of recent ACI Papers and Reports

Institute papers of this JOURNAL Volume 20 are currently available at prices indicated. Please order by title and title number.

RECOMMENDED PRACTICE FOR WINTER CONCRETING METHODS (ACI 604-48).....45-1

Price 50 cents.

REPORT of COMMITTEE 604—Sept. 1948, pp. 1-20, (V. 45) in special covers

Supersedes 44-13.

This ACI standard establishes methods of cold-weather concreting for thin sections and mass concrete. Heating of materials, accelerators and anti-freezes, curing and temperature records during curing, subgrade (or base) preparation, protective coverings during curing, and form removal are discussed for both types of job, and preferred methods are indicated. An appendix entry outlines objectives of the special winter methods with background material which indicates the "why" of some of the recommended practices. Charts in the appendix indicate effect of curing temperature on concrete strength, and a list of 135 selected references to periodical literature on winter concreting methods is included.

RELATIVE STRENGTHS OF PORTLAND CEMENT MORTAR IN BENDING UNDER VARIOUS LOADING CONDITIONS.....45-2

Price 35 cents.

JACOB PORTER FRANKEL—Sept. 1948, pp. 21-32 (V. 45)

Assuming the behavior of standard mortar to be similar to that of plain concrete, tests were performed on 99 small mortar beams under sixth-, third- and center-point loading to verify the applicability of the statistical theory of the strength of brittle materials to concrete structures. The theory, as first developed by Weibull, is briefly analyzed, compared to the methods used by Tucker, and finally applied to the tests at hand. Agreement between the theoretical and experimental findings is so close as to warrant future testing on plain concrete specimens.

TERRAZZO AS AFFECTED BY CLEANING MATERIALS.....45-3

Price 35 cents.

D. W. KESSLER—Sept. 1948, pp. 33-40 (V. 45)

Effect of cleaning materials on terrazzo was studied using solutions of the following detergents: soda ash, trisodium phosphate and synthetic sulfonate. Seventy-six different aggregates were used in preparing terrazzo disks which were moist cured 3 months and then surface ground to form a shallow dish. Detergent was allowed to stand in dish 30 minutes before rinsing and drying (at 105 C). This testing cycle was repeated until deterioration tendencies were established. Soda ash solution proved much more destructive than the trisodium phosphate; however, up to the time of writing (after 850 cycles) the synthetic sulfonate had caused no failure.

TRANSPORTING READY-MIXED CONCRETE IN OPEN DUMP TRUCKS.....45-4

Price 35 cents.

R. A. BURMEISTER—Sept. 1948, pp. 41-56 (V. 45).

Milwaukee experience in hauling air-entraining concrete in open dump truck bodies is recorded. Slump, air content, specific weight, compressive strength and workability tests were made on concrete before and after a trial run over bumpy pavement. Some similar tests were made on concrete being placed in a Milwaukee street after open truck transportation. The author concludes that open trucks for hauling ready-mixed air-entraining concrete are satisfactory when air content is between 3 and 6 percent (5 to 6 sacks cement per cu yd) and hauling time is 45 minutes or less, provided that aggregate moisture, grading and slump are carefully controlled.

A STUDY OF ALKALI-AGGREGATE REACTIVITY BY MEANS OF MORTAR BAR EXPANSIONS.....45-5

Price 35 cents.

T. M. KELLY, L. SCHUMAN and F. B. HORNIBROOK—Sept. 1948, pp. 57—80 (V. 45)

Many types of aggregate were combined in varying amounts and sizes with high- and low-alkali cements and formed into 1 x 1 x 10-in. mortar bars. The bars were stored either at 70 F or at 100 F and their expansions measured at ages ranging from 1 month to 4 years.

In combination with high-alkali cements, opal, opaline chert and a siliceous dolomitic limestone were found to cause greatest expansion. Certain aggregates containing volcanic glasses and some natural sands and gravel also caused excessive expansion; with one exception, these sands contained small amounts of opal.

Greatly delayed expansion resulted with the very fine sizes of opal, particularly in combination with high-soda cement. Similar behavior resulted with minus No. 81 size opal and low-alkali cement with either Na_2SO_4 or K_2SO_4 additions.

Materials such as dehydrated kaolin, soda feldspar, magnesium fluosilicate, acetic acid and calcium hydroxide added in small amounts as correctives were ineffective. However, diatomaceous earth in sufficient quantity as a cement replacement eliminated expansion.

REINFORCED CONCRETE WALL AND COLUMN FOOTINGS

PART I.....45-6a

Price 60 cents.

FRANK E. RICHART—Oct. 1948, pp. 97-128 (V. 45)

PART II.....45-6b

Price 35 cents.

FRANK E. RICHART—Nov. 1948, pp. 237-260 (V. 45)

This paper presents, in two parts, a report on an experimental investigation of reinforced concrete footings. It represents the first extensive study of the subject since the well-known work of Talbot in 1913. In the meantime there have been many developments in materials, as well as in design methods.

Important developments from these tests include the following: (a) Definite proof that the tensile and bond resistance depend upon the bending moment and shear found by statics by consideration of the full applied load, and not 85 percent thereof, as assumed in current building codes; (b) Hooked ends of bars showed no particular advantage in bond resistance over straight bars, particularly when deformed bars of new improved types were used; (c) Hooked bars produced very little effect on resistance to diagonal tension, much less than is commonly assumed; (d) Welded mats proved particularly effective in resisting end slip of bars; (e) Footing caps or piers are effective in reducing the amount of reinforcement and in increasing the load capacity of footings; and (f) Diagonal tension seems to be the point of weakness in current design practice. The factor of safety of thin footings in this respect appears greater than in thick footings, and is generally greater in rectangular than in square footings when the conventional methods of computation are used.

In these tests, major emphasis has been placed on the isolated column footing. Principal attention has been

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The AMERICAN CONCRETE INSTITUTE

is a non-profit, non-partisan organization of engineers, scientists, builders, manufacturers and representatives of industries associated in their technical interest with the field of concrete, dedicated to the public service. Its primary objective is to assist its members and the engineering profession generally, by gathering and disseminating information about the properties and applications of concrete and reinforced concrete and their constituent materials.

Since 1905 that primary objective has been achieved by the combined membership effort. Individually and through committees, and with the cooperation of many public and private agencies, members have correlated the results of research, from both field and laboratory, and of practices in design, construction and manufacture.

The work of the Institute has become available to the engineering profession in annual volumes of ACI Proceedings since 1905. Beginning 1929 the Proceedings have first appeared periodically in the Journal of the American Concrete Institute and in many separate publications.

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given to the resistance of footings to failure by bond, diagonal tension and tension in the steel. Test footings were designed to produce these various types of failure. In addition, studies were made of the behavior of rectangular footings and footings having intermediate capitals or piers.

The tests featured measurements of tensile and compressive strains, deflections and slippage of bars. In some of the tests three grades of concrete were used, in others, five types of reinforcing bar were employed. In most of the tests, the footings were 7 ft square. They were supported on a bed of steel springs and tested in a large testing machine.

This paper merely attempts to present and interpret the test results, and does not offer recommendations as to possible changes in design procedures at this time.

LATTICE ANALOGY IN CONCRETE DESIGN.....45-7

Price 35 cents.

DOUGLAS McHENRY—Oct. 1948, pp. 129-140 (V. 45)

The lattice analogy is a scheme for solving two-dimensional stress problems in which the involved mathematical methods of the theory of elasticity are replaced by simple computations. The solid section is replaced by an equivalent lattice or framework which may be solved by methods applicable to indeterminate structures. Stress distribution in sections of complex shape and with complex loading may be determined by successive approximations which involve only substitution in simple equations. The method is illustrated by application to the problem of stresses in a deep beam with off-center loading.

STRENGTH OF PRECAST CONCRETE FLOOR JOISTS.....45-8

Price 35 cents.

JACOB FELD—Oct. 1948, pp. 141-148 (V. 45)

The strength and usability of precast concrete floor joists left in the open without any protection for a year, when a housing project was abandoned, were proved by load test on a slab section incorporating the poorest joists. This paper describes the condition of the joists, the load test results and indicates that a greater tolerance can be safely permitted in the visible defects of precast concrete joists.

EFFECT OF ENTRAINED AIR ON CONCRETES MADE WITH SO-CALLED "SAND-GRAVEL" AGGREGATES.....45-9

Price 35 cents.

PAUL KLIEGER—Oct. 1948, pp. 149-164 (V. 45)

Sand-gravel aggregates (maximum particle size ⅜ in.) used with non-air-entraining cement produce concretes containing from 3 to 4 percent air. The same mixes made with air-entraining cement and the sand-gravel aggregates have air contents from 8 to 13 percent. Effect of this larger amount of entrained air is given in terms of tests of flexural and compressive strength, freezing and thawing resistance, and length changes in varying storage conditions. Effect of "sweetening" the sand-gravel by addition of 30 percent of coarser aggregate is noted.

BURNED SHALE AND EXPANDED SLAG CONCRETES WITH AND WITHOUT AIR-ENTRAINING ADMIXTURE.....45-10

Price 35 cents.

P. H. PETERSEN—Oct. 1948, pp. 165-176 (V. 45)

The physical properties of several lightweight aggregate portland cement concretes made with burned shale or expanded slag were investigated at the National Bureau of Standards. Three grades of concrete were made with each aggregate. Air-entrainment greater than 20 percent is reported for the mixtures leanest in cement, an air-entraining admixture being used to increase the workability of all but the richest concretes. Compressive, flexural and bond strength data are given as well as resistance to heat transfer, rain penetration and water penetration by capillarity. Also included are the coefficients of thermal expansion, shrinkage, and values for change in length due to wetting and drying.

CEMENSTONE PRECAST CONSTRUCTION.....45-11

Price 35 cents.

A. C. GRAFFLIN—Nov. 1948, pp. 193-204 (V. 45)

Standardized precast reinforced concrete structural members and panels point toward economy in fireproof construction of ordinary buildings up to four stories. The method described has been so developed that architects and engineers can select from tables reinforced concrete beams, columns, roof, floor, and wall panels in the same manner as for structural steel. Design is in accordance with the ACI Code and costs compare favorably with similar structures framed of steel and fireproofed with concrete.

SAND GRADING INFLUENCE ON AIR ENTRAINMENT IN CONCRETE.45-12

Price 35 cents.

M. A. CRAVEN—Nov. 1948, pp. 205-216 (V. 45)

Four series of mixes with varying air-entraining agents, cement factors, and sand grading and content were prepared in order to observe sand grading effect on concrete. Graphic record is presented of air content and W/C plotted against fineness modulus of sand; flow and compressive strength plotted against fineness modulus of sand; and air content plotted against percent of No. 30—No. 50 sand and total sand percentage. Generally the percentage of air entrained in concrete increased with decrease in fineness modulus of sand. Quantity of air appears to be a function of the quantity of No. 30—No. 50 sand. Effect of grading and quantity of sand on other properties of fresh and hardened concrete is noted.

INFLUENCE OF SIZE GRADING OF SAND ON AIR ENTRAINMENT.45-13

Price 35 cents.

E. W. SCRIPTURE, Jr., F. B. HORNIBROOK, and D. E. BRYANT—Nov. 1948, pp. 217-228 (V. 45)

Field reports indicated difficulty with certain sands in securing the desired amount of entrained air; this was frequently attributed to the size grading of the sand, particularly to a deficiency in the finer fractions. An experimental investigation was undertaken in view of the paucity of published data on this subject. Mixes of sand and water alone, 1:4 and 1:2 mortars, and concrete mixes were made with and without air-entraining agents and the air contents determined. While size grading of the sand had a very large influence on air entrainment in a mixture of sand and water alone, this effect was smaller in mortars and very small in concrete mixes. It was found that maximum air was entrained by the 28-48 mesh size sand rather than the 48-100 mesh size.

USE OF PREPACKED AGGREGATE CONCRETE IN MAJOR DAM CONSTRUCTION.....45-14

Price 35 cents.

PAUL BAUMANN—Nov. 1948, pp. 229-236 (V. 45)

This paper sets forth the method of construction of a major gravity dam whereby the average temperature of the mass concrete may be controlled within the mean annual and the dry shrinkage reduced to a fraction of that of conventional concrete by means of prepacking of the coarse aggregate to a density corresponding to 30 percent voids; precooling the aggregate; the intrusion of the coarse aggregate by mortar with a cement content of two sacks per cu yd or less; and the internal post-cooling, curing and draining of the mass concrete. (This process is known as the Prepakt method of construction.)

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ACI publications in large current demand

Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-48)

Approved as a Standard of the American Concrete Institute at its 44th Annual Convention February 25, 1948. Ratified by letter ballot June 29, 1948. It is of large format, bound to lie flat and presents typical engineering and placing drawings with discussion calling attention to important considerations in designing practice. It was prepared to simplify, speed, and effect standardization in detailing. It is believed to be the only publication of its kind in English. It is meeting wide acclaim among designers, draftsmen and in engineering schools. Price—$3.00, to ACI Members—$1.75.

ACI Standards—1948

202 pages, 6x9 reprinting current ACI Standards: Building Code Requirements for Reinforced Concrete (ACI 318-47); Minimum Standard Requirements for Precast Concrete Floor Units (ACI 711-46); five recommended practices: Use of Metal Supports for Reinforcement (ACI 319-42); Measuring, Mixing and Placing Concrete (ACI 614-42); Design of Concrete Mixes (ACI 613-44); Construction of Concrete Farm Silos (ACI 714-46); Winter Concreting Methods (ACI 604-48); and two specifications: Concrete Pavements and Bases (ACI 617-44) and Cast Stone (ACI 704-44)—nine ACI Standards in one book, $2.00 per copy—to ACI Members, $1.25. The "Detailing Manual" (ACI 315-48) is available separately only at its established price—see above.

Air Entrainment in Concrete (1944)

92 pages of reports of laboratory data and field experience including a 31-page paper by H. F. Gonnerman, "Tests of Concretes Containing Air-entraining Portland Cements or Air-entraining Materials Added to Batch at Mixer," and 61 pages of the contributions of 15 participants in a 1944 ACI Convention Symposium, "Concretes Containing Air-entraining Agents," reprinted (in special covers) from the ACI JOURNAL for June, 1944, $1.25 per copy, 75 cents to members.

Air Entrainment in Concrete—Book 2 (1947)

A 204-page compilation of information on air entrainment published after Book 1 (1944), including: "Field Use of Cement Containing Vinsol Resin", and "Laboratory Studies of Concrete Containing Air-Entraining Admixtures", by Charles E. Wuerpel, "Entrained Air in Concrete", a foreword and 14 short papers presented at the 1946 Convention; and discussion of the symposium, reprinted from ACI JOURNALS for September 1945, Feb., June and Dec. Part 2, 1946. $2.25 per copy, $1.50 to ACI Members.

ACI Manual of Concrete Inspection (July 1941)

This 140-page book (pocket size) is the work of ACI Committee 611, Inspection of Concrete. It sets up what good practice requires of concrete inspectors and a background of information on the "why" of such good practice. Price $1.00—to ACI members 75 cents.

"The Joint Committee Report" (June 1940)

The Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete submitting "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete," represents the ten-year work of the third Joint Committee, consisting of affiliated committees of the American Concrete Institute, American Institute of Architects, American Railway Engineering Association, American Society of Civil Engineers, American Society for Testing Materials, Portland Cement Association. Published June 15, 1940, 140 pages. Price $1.50—to ACI members $1.00.

Reinforced Concrete Design Handbook (Dec. 1939)

This report of ACI Committee 317 is in increasing demand. From the Committee's Foreword: "One of the important objectives of the committee has been to prepare tables covering as large a range of unit stresses as may be met in general practice. A second and equally important aim has been to reduce the design of members under combined bending and axial load to the same simple form as is used in the solution of common flexural problems."—132 pages, price $2.00—$1.00 to ACI members.

For further information about ACI Membership and Publications (including pamphlets presenting synopses of recent ACI papers and reports) address:

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RATIONAL ANALYSIS AND DESIGN OF TWO-WAY CONCRETE SLABS.....45-15

Price 60 cents.

C. P. SIESS and N. M. NEWMARK—Dec. 1948, pp. 272-316 (V. 45)

A new method for the design of two-way building slabs is proposed and its step-by-step development is described. A new moment distribution procedure is used to compute moments in a number of rectangular slabs continuous over rigid beams. Several variables are studied in the analyses and include: the ratio of sides, the effect of discontinuous edges, the torsional stiffness of the beam, various types of loading, and combinations of panels of various sizes and shapes. Certain conclusions are stated regarding the types of loading to be considered and the values of beam torsional stiffness to be assumed in the development of the design procedure.

Moments obtained in the foregoing analyses are modified to take into account the effect of additional variables. First, the slab moments are increased by various amounts as a result of the deflection of the beams. Next, separate coefficients for dead and live load are replaced by combined coefficients for a live load-dead load ratio of 3.0. Then follows a study to determine the effect of discontinuous edges, and this variable is eliminated from the procedure. Finally all slab moments are reduced by 20 percent in recognition of the redistribution of moments at high load. Additional studies are then made of the distribution of moments across the width of the slab and of the moments to be used for the design of the beams to complete the development of the design procedure.

A proposed design procedure is presented in specification form and is compared with other procedures of a similar nature and with the moment distribution procedure described in the appendix.

CORROSION RESISTANCE TESTS OF CONCRETE FLOORS—WITH AND WITHOUT METALLIC AGGREGATE.....45-16

Price 35 cents.

E. W. SCRIPTURE, JR. and C. H. SAKRYD—Dec. 1948 pp. 317-324 (V. 45)

An attempt has been made to determine the conditions under which concrete floors can be expected to give good service. Resistance to abrasion was determined on specimens with and without metallic aggregate after exposure to typical reagents selected from the groups described by the Joint Committee as having corrosive effects of an intermediate nature between no attack and severe attack. On the basis of the experimental results the reagents in the Joint Committee report have been classified in relation to anticipated service of concrete floors.

FLEXICORE PRECAST FLOOR AND ROOF SLABS.....45-17

Price 35 cents.

GAYLE B. PRICE—Dec. 1948, pp. 325-340 (V. 45).

This paper describes briefly the design, manufacture, erection procedures and use of Flexicore slabs in precast concrete floor and roof systems. The Flexicore slab has a standard cross section 12 in. wide and 6 in. thick, lightened by two holes 4⅝ in. in diameter running the full length of the slab which is made in inch variations of length up to a maximum of 22 ft 6 in. Reinforcement consists of two bars in the top of the slab for "handling" stresses and two or three prestressed tension bars at the bottom to take care of bending moment.

PROPOSED RECOMMENDED PRACTICE FOR THE APPLICATION OF PORTLAND CEMENT PAINT TO CONCRETE SURFACES.....45-18

Price 35 cents.

REPORT of COMMITTEE 616—Jan. 1949, pp. 353-368, (V. 45)

This report is a studied revision by a reorganized committee of the previous report of Committee 616 published in June 1942, based upon criticisms of that report plus information from more recent tests and experiences. Included are recommendations for appropriate usage, age of concrete, preparation of surface, and the preparation, application and curing of portland cement paint. Three appendixes discuss composition, manufacture and storage and, general characteristics and factors affecting durability.

DRY MORTAR AS A BEARING AND GROUTING MATERIAL.....45-19

Price 35 cents.

BOYD S. BROOKS—Jan. 1949, pp. 369-380, (V. 45)

In 1939, at the navy's new David Taylor Model Basin at Carderock, Md., there was an urgent need for a non-shrinking bearing material to be used under the cast-steel "chairs" which support the rails on which the towing carriages run. Requests for information concerning materials and installation procedures produced such varied answers that investigation was started to find the best material and the best procedures for placing it.

The bearing material selected for use was 1:3 portland cement mortar, with 4.13 gallons of water per sack of cement. This was rammed firmly into place.

This bearing material has now been in place over a period of about 9 years under extremely severe conditions and without exception has proved to be excellent in function and condition.

DIRECT DIMENSIONING OF RECTANGULAR SECTIONS.....45-20

Price 35 cents.

MICHEL BAKHOUM—Jan. 1949, pp. 381-396 (V. 45)

On the basis of the standard theory for reinforced concrete design, methods have been derived for the direct dimensioning of rectangular sections, with or without compression reinforcement, when subjected to simple bending, eccentric compression or eccentric tension. Two cases are considered: when the depth of the section is fixed and when it is not fixed. The conditions leading to minimum reinforcement are also studied. The solutions developed have been simplified by the use of curves which are applicable for pure bending as well as for eccentric compressive or tensile forces and cover any values o f_c, f_s and n. The curves allow making a variety of designs in a relatively short time and thus help in choosing the most economical or most suitable section.

STRENGTHENING BRIDGE SLABS WITH GROUTED REINFORCEMENT.45-21

Price 35 cents.

S. O. ASPLUND—Jan. 1949, pp. 397-408

On the project described the negative reinforcing bars settled as much as 2½ in. out of correct position. Various means for incorporating negative reinforcement at correct height in the finished structure are discussed. The method selected of grouting additional bars in grooves cut with the aid of diamond saw is described. Tests on beam specimens made with some bars encased in the concrete and some bars grouted into diamond-sawed grooves, give results identical with those to be expected for all bars normally encased; ultimate strengths conform closely to the plastic bending capacity of reinforced concrete according to design methods originated by C. S. Whitney.

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Current ACI Standards

Building Code Requirements for Reinforced Concrete (ACI 318-47)
64 pages in covers: 50 cents per copy (40 cents to ACI Members)

Minimum Standard Requirements for Precast Concrete Floor Units (ACI 711-46)
16 pages in covers: 50 cents per copy (40 cents to ACI Members)

Recommended Practice for the Construction of Concrete Farm Silos (ACI 714-46)
16 pages in covers: 50 cents per copy (40 cents to ACI Members)

Recommended Practice for the Design of Concrete Mixes (ACI 613-44)
24 pages in covers: 50 cents per copy (40 cents to ACI Members)

Specifications for Cast Stone (ACI 704-44)
4 pages: 35 cents per copy

Specifications for Concrete Pavements and Bases (ACI 617-44)
30 pages in covers: 50 cents per copy (40 cents to ACI Members)

Recommended Practice for Measuring, Mixing and Placing Concrete (ACI 614-42)
30 pages in covers: 50 cents per copy (40 cents to ACI Members)

Recommended Practice for the Use of Metal Supports for Reinforcement (ACI 319-42)
4 pages: 35 cents per copy

Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-48)
A publication of large format, bound to lie flat.
55 pages: $3.00 per copy ($1.75 to ACI Members)

Recommended Practice for Winter Concreting Methods (ACI 604-48)
20 pages in covers: 50 cents per copy (40 cents to ACI Members)

Recent Proposed ACI Standards

Proposed Recommended Stresses for Unreinforced Concrete
Reported by Committee 322 as information and for discussion only. 4 pages, 35 cents per copy. (Reprint from ACI JOURNAL, Nov. 1942)

Proposed Recommended Practice for the Application of Portland Cement Paint to Concrete Surfaces
Released by Committee 616 and the Standards Committee for publication for consideration for adoption as a standard. 16 pages, 35 cents per copy (Reprint from ACI JOURNAL, Jan. 1949)

Continued from p. 46

COMPARATIVE DESIGNS OF A SEGMENTAL SKEWED FRAME CONCRETE BRIDGE BY THE STRAIGHT LINE AND PLASTIC THEORY METHODS.....45-22

Price 35 cents.

MILTON BRUMER—Jan. 1949, pp. 409-420 (V. 45)

The designs of a segmental skewed frame concrete bridge are discussed and compared as developed by both the straight line and plastic theory methods. The plastic theory will lead to rigid concrete frames of more economical proportions capable of sustaining equal or greater loads than frames designed by the straight line method. Effects of volume changes and yielding supports are also of smaller consequence. Deflections may be expected to be greater. The author concludes that further substantiation of the plastic theory is desirable.

SOME FACTORS AFFECTING AIR ENTRAINMENT.....45-23

Price 35 cents.

E. W. SCRIPTURE, Jr. and F. J. LITWINOWICZ—Feb. 1949, pp. 433-444 (V. 45).

A program was undertaken to investigate the influence of various factors on the amount of air entrained in concrete mixes with and without air-entraining agent added. The variables so far studied are slump, cement factor, and sand-total aggregate ratio. The results indicate that the two latter factors are of major importance but that slump has less effect. In general, entrained air content increases with increasing slump, decreases with increasing cement factor, sand-total aggregate ratio.

EXPERIMENTAL AIDS IN STRUCTURAL CONCRETE DESIGN...45-24

Price 35 cents.

R. E. GLOVER, O. J. OLSEN and CARL ZANGAR—Feb. 1949, pp. 445-468 (V. 45).

Statically determinate and indeterminate structures are generally designed by analytical methods. This paper describes several experimental aids which have been used in the design and stress analysis of statically indeterminate structures at the laboratories of the Bureau of Reclamation. These methods are used where analytical procedures cannot conveniently be applied. Described herein are photoelastic methods using both the polariscope and interferometer, the Beggs Deformeter, Stresscoat and electric SR-4 strain gages. Applications are given for each method.

PRACTICES, EXPERIENCES, AND TESTS WITH AIR-ENTRAINING AGENTS IN MAKING DURABLE CONCRETE.....45-25

Price 35 cents.

R. F. BLANKS and W. A. CORDON—Feb. 1949, pp. 469-488 (V. 45)

The authors discuss the advantages of air entrainment with regard to durability, permeability, workability, reduction in alkali expansion, time saving, reduction in water and cement, temperature rise, strength, abrasion resistance, monolithic lightweight concrete and mass concrete. U. S. Bureau of Reclamation experiences are discussed with regard to pumping, transportation and placing, grading of aggregates, factors affecting the amount of air entrained Air-entraining agents and air-entraining cements are discussed briefly. There is a brief section on the measurements of entrained air in concrete with a recommended procedure.

CONCRETE FLOOR FINISHING.....45-26

Price 35 cents.

GERALD MILSOM—Feb. 1949, pp. 489-492 (V. 45).

A concise discussion of the requirements for durable concrete floors including careful selection and control of materials and attention to weather conditions. The importance of proper equipment, working conditions and curing of the finished concrete are emphasized.

PLASTIC FLOW OF CONCRETE AT HIGH OVERLOAD.....45-27

Price 35 cents.

J. R. SHANK—Feb. 1949, pp. 493-500 (V. 45).

New data on the plastic flow of concrete prompted a reconsideration of the assumption that plastic flow is directly proportional to the unit stress applied. Tests are described in which deformations of 6 x 12-in. concrete cylinders were measured while under sustained loads approaching the ultimate strength. Plotted data are shown to demonstrate that the change in the rate of plastic flow appears to be uniform up to three-quarters or more of the ultimate strength with an abrupt change at the "true" ultimate strength. Included are other data indicating that the plastic flow curve has the same general form independent of time and applied load leading to a new consideration in expressing factor of safety and developing beam theories on a factor of safety basis.

NOTICE!

Copies of original articles and books reviewed in the Current Reviews section of the JOURNAL are not available from ACI, The footnote formerly published with that section indicated that separate prints of *the reviews section itself* are available, not separate prints of the original articles reviewed, as some readers may have assumed.

Separate prints of current ACI papers and reports are available at the prices listed in the adjoining synopses. Send your requests for these and other information to:

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DISCUSSION

Discussion closed January 1, 1949

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Recommended Practice for Winter Concreting Methods (ACI 604-48)—Report of Committee 604

Relative Strengths of Portland Cement Mortar in Bending Under Various Loading Conditions—Jacob Porter Frankel

Terrazzo as Affected by Cleaning Materials—D. W. Kessler

Transporting Ready-Mixed Concrete in Open Dump Trucks—R. A. Burmeister

A Study of Alkali-Aggregate Reactivity by Means of Mortar Bar Expansions—T. M. Kelly, L. Schuman, F. B. Hornibrook

Discussion closed February 1, 1949

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Lattice Analogy in Concrete Design—Douglas McHenry

Strength of Precast Concrete Floor Joists—Jacob Feld

Effect of Entrained Air on Concretes Made With So-Called "Sand-Gravel" Aggregates—Paul Klieger

Burned Shale and Expanded Slag Concretes With and Without Air-Entraining Admixture—P. H. Petersen

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Reinforced Concrete Wall and Column Footings, Parts 1 and 2—Frank E. Richart

Cemenstone Precast Construction—A. C. Grafflin

Sand Grading Influence on Air Entrainment in Concrete—M. A. Craven

Influence of Size Grading of Sand on Air Entrainment—E. W. Scripture, Jr., F. B. Hornibrook and D. E. Bryant

Use of Prepacked Aggregate Concrete in Major Dam Construction—Paul Baumann

Discussion closes April 1, 1949

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(ACI PROCEEDINGS Vol. 45)

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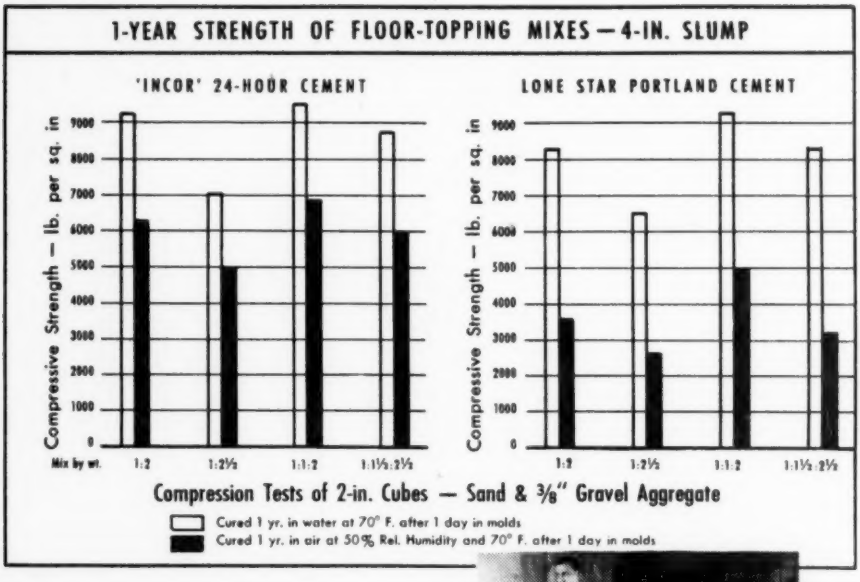

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Papers, committee reports and other contributions to the **Proceedings** pages of this Journal, as submitted in furtherance of Institute objectives, are, if published, accepted on the sole responsibility of their authors. Institute authority attaches only to standards adopted as provided in the By-Laws. All manuscripts prepared with a view to Journal publication should be submitted in triplicate, addressed: Secretary, Technical Activities Committee, American Concrete Institute, New Center Building, Detroit 2, Michigan.

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Journal subscription price $10.00 per year payable in advance. Members receive the Journal as one of the considerations of dues payment. The Journal is issued monthly, January to June and September to December (the volume year September to June being completed by title page, contents, indexes and closing discussion in Part 2 of the subsequent December issue).

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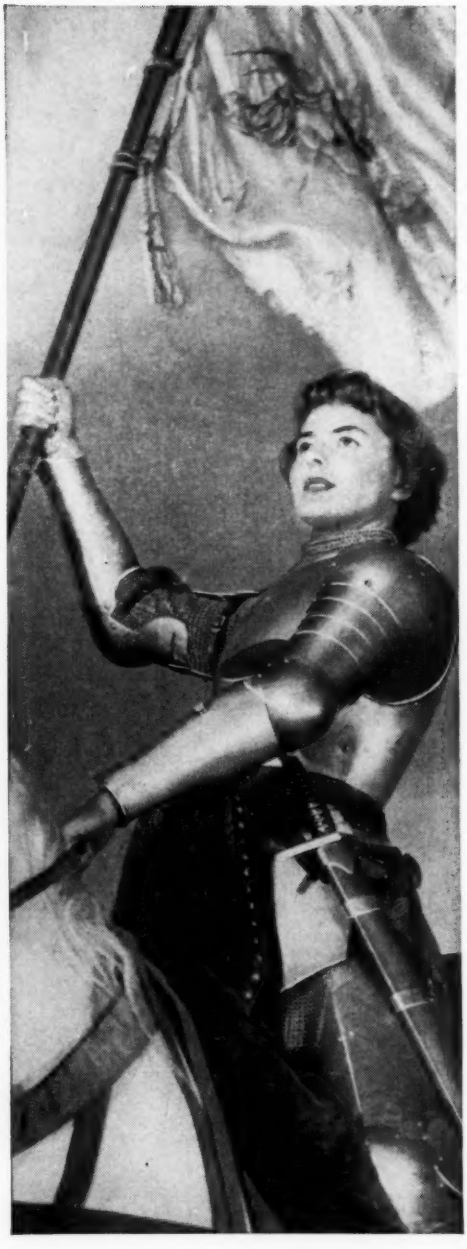

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POLICY

It is the policy of the American Concrete Institute to encourage participation by its members and others in the work of extending the knowledge of concrete and reinforced concrete as a basis for improved products and structures.

To this end the Board of Direction has assigned to the Technical Activities Committee the responsibility of selecting for publication such papers, committee reports, discussions and other contributions or parts of such contributions as, in the judgment of the Committee, seem to offer most of value in attaining Institute objectives within space requirements, consistent with budget limitations.

The New York Convention—just over as this page goes to press—saw a turnout of 528. See April News Letter for a review of convention events.

Prof. Herbert J. Gilkey, Iowa State College, Ames, Iowa, was elected President for the coming year.

Harry F. Thomson became a Vice President; Harmer E. Davis, H. C. Delzell, N. M. Newmark, R. W. Spencer new directors; and Henry L. Kennedy and Paul W. Norton were reappointed to the Board for the unexpired terms of A. J. Boase and Mr. Thomson.

The 1950 ACI convention is scheduled for the Edgewater Beach Hotel, Chicago, week of February 20.

In February ACI received 126 new member applications, which if approved by the Board of Direction will result in a net gain of 97 and a new total membership of 4,585 as of March 1. It is of interest that January applications also totaled 126.

A third printing again makes available "Manual of Standard Practice for Detailing Reinforced Concrete Structures." Price—$3.00, to ACI members $1.75.

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Recent Proposed ACI Standards

Proposed Recommended Stresses for Unreinforced Concrete
Reported by Committee 322 as information and for discussion only. 4 pages, 35 cents per copy. (Reprint from ACI JOURNAL, Nov. 1942)

Proposed Recommended Practice for the Application of Portland Cement Paint to Concrete Surfaces
Adopted as a standard by the 45th annual ACI convention February 24, 1949 subject to ratification of membership by letter ballot to be canvassed about July 1, 1949. 16 pages, 35 cents per copy (Reprint from ACI JOURNAL, Jan. 1949)

The transition of highway use from occasional fair-weather driving to a vital link in the transportation system of the country has necessitated, in the north, the winter application of chemicals for ice control with damaging effect on portland cement concrete pavements. How a discerning field observation was translated into a laboratory test program resulting in a pavement treatment procedure to prevent such damage is the main thread of Mr. Tallamy's paper.

Control of Concrete Pavement Scaling Caused by Chloride Salts*

By B. D. TALLAMY†

SYNOPSIS

Under modern traffic requirements on heavily traveled roads salt-treated abrasives will not remove ice quickly enough. As maintenance forces have struggled to meet the demand for uninterrupted service the use of straight salts has become increasingly common. Direct applications of up to 600 lb of salt per two-lane mile are required to combat extreme icing conditions. Pavements constructed to withstand the weak brines deposited by salt-treated abrasives cannot stand up under straight salt. New pavements appear more vulnerable than those two or more years old.

The observed resistance to salt action of the oil soaked center streak of uphill traffic lanes led New York state highway engineers to investigate the feasibility of the use of dilute oil applications as a protective measure. Laboratory tests indicate successful results may be expected. In the late summer of 1948 oil spray equipment was developed in time to treat nearly 60 miles of new concrete highways, which should provide a wide base for field observation of the effectiveness of the method.

INTRODUCTION

When the economy of a people comes to be dependent upon the highway system that serves them, those transportation arteries must be maintained at all times. Night and day, winter or summer, despite natural forces which may threaten, the vital flow of goods and people over the highway system must move safely and on time. The struggle to project adequate highways into every inhabited corner of the nation has developed highway engineering to the point where it is equal to almost any challenge. However, the insistence of the engineer on keeping the highways open at all times has met some formidable resistance from nature. His efforts to defy the weather have run head on into

*Presented at the ACI 45th annual convention, New York, New York, February 23, 1949. Title No. 45-28 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE V. 20, No. 7, Mar. 1949. *Proceedings* V. 45. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than July 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

†Superintendent of Public Works, State of New York, Albany.

conditions which seriously threaten the life and serviceability of the very materials with which he builds.

The most vital problem of this kind is that of ice control. At present the only known practical method of combatting traffic hazards caused by ice and freezing rain is to apply chemicals in sufficient quantity to quickly break up the deposits. For a number of years sodium chloride and calcium chloride have been used throughout the northern states where icing conditions are frequent during the winter months. On concrete pavements, however, chloride salts have caused scaling and disintegration in varying degrees depending upon the number and frequency of applications, the quantity used, type and age of concrete, and other variables.

Much effort has been expended to counteract these deleterious effects on concrete pavements by using various admixtures, selective blending and grading of ingredients, and protective coatings, either singly or in combination. After working diligently through all these phases, the New York State Department of Public Works recently arrived at what is hoped to be a major step toward the solution of the problem.

ICE CONTROL PRACTICES

Salt-abrasive mixtures

For the past generation it has been rather common practice throughout the northern states, including New York, to combat icing conditions on highways by spreading on them sand or other abrasives to which chemicals had been added. The earlier admixture was calcium chloride. Later sodium chloride, or common salt, was found to be equally effective. It was soon determined that the addition of 50 to 75 lb of salt per cu yd of abrasives previously stock-piled at convenient points along the roadside produced the desired characteristics. First, the distribution of salt through the abrasive kept stock piles from freezing. Second, the salt-coated particles of abrasive, when distributed over the pavement, softened the ice surface and became imbedded in it. Ultimately, the mass of ice weakened and broke under traffic.

Eventually, after application of salt-treated abrasives, weak salt brine percolated through to the pavement; and it soon became evident that straight portland cement concrete could not withstand the chemical action of chloride salts. Scaling and surface disintegration of concrete pavements followed where sanding was necessary to combat ice.

In 1936, the New York State Department of Public Works advanced its attack on scaling by the construction of a series of experimental roads composed of concretes in which the cements were fortified by various admixtures. These admixtures, ground into the cement at the mills, included tallow, fish oil, and Vinsol resin. During the same year, another

series of experimental roads were constructed with varying proportions of standard portland cements and natural cement.

These proved rather successful, and as a result concrete pavement constructed subsequently with natural cement blends and with air entraining agents were able to withstand the use of salt-treated abrasives for ice control.

Straight salt treatment

On frequent occasions, the application of treated abrasives is not adequate to keep modern vehicles and dense traffic moving properly during severe winter conditions. Many present truck delivery systems move on schedules whose accuracy approach that of crack railroad trains. The businessman, trucker, and all the other millions of citizen-taxpayers have every right to expect the maintenance forces to have the roads in safe, usable condition.

As highway maintenance departments have struggled to meet this present-day demand for uninterrupted service, the practice of applying straight sodium chloride or calcium chloride to slippery pavements has become increasingly common. Direct applications, ranging upward to 600 pounds of salt per two-lane mile, are required to combat extreme icing conditions. In some instances, such applications may leave deposited on the pavements saturated solutions containing 30 percent of salt at zero temperatures. It is, then, small wonder that concrete pavements exposed to such punishing doses react unfavorably to them, especially when newly constructed. Pavements constructed to safely withstand the weak brines deposited by salt-protected abrasives cannot stand up under straight salt. New pavements appear to be much more vulnerable than those which have weathered two or more years.

LABORATORY INVESTIGATIONS

The laboratory attacked anew the problem of pavement scaling which had been solved to some degree ten years before. The outlook was discouraging for it was felt that methods of fortifying the concrete itself by admixtures had been exhausted. Earlier attempts to protect the pavement by surface applications of bituminous compounds, asphalt, tar, and linseed oil seemed to have failed. These surface coatings dried out, chipped, or wore off much the same as paint might.

Then, in 1947, came the first break in solving the problem. Somewhat after the pattern of a detective novel, what we now hope to be a satisfactory solution was found at the scene of the crime. Scaling is most pronounced on steep hills, long grades, intersections, and other places where the heaviest and most frequent applications of pure chemicals usually are made. At such locations, particularly on the up-hill lanes

where streaks of oil drippings were heaviest, there was a virtual absence of scaling or disintegration of the concrete. Climbing steep grades, engines of motor cars and trucks throw off more oil than they do on level stretches of highway producing a characteristic dark streak at the center of the lane. On the down-hill lanes where less oil is normally deposited, damage due to concentrated salt solutions was much more readily apparent. From that observation it was a simple step to reason that, if oil, dropped accidentally, protected the concrete pavement, then oil deposited purposely should accomplish the same result.

Test procedure

This theory was put to test in the laboratory without delay. The object was to determine the effectiveness of motor oil in resisting the action of salt and to find the minimum quantity of oil that would be needed to provide adequate protection. The entire series of tests was conducted on 2-in. mortar cubes which would be easy to make and handle. All cubes were made with three parts Long Island sand and one part of cement. Cubes of straight portland cement mortar were adopted as the control or standard of comparison because of the known vulnerability of portland cement concrete even to weak brines. Other cubes were made with a blend of one part natural cement with six parts of portland cement. Approximately 200 cubes were studied in the series of experiments. All were cured equally in the moist air cabinet.

SAE 10 motor oil was used throughout the tests because its viscosity was quite comparable to the gasoline-diluted drippings from motor vehicles, the action of which the laboratory was trying to duplicate. Twelve different treatments were given sets of both straight portland and natural-portland cubes. These treatments included one, two, and three brush-applied coats of straight oil; one, two, and three brush-applied coats of oil diluted with gasoline in three ratios; 90 to 10, 75 to 25, and 50 to 50.

The diluted oil mixtures were introduced to facilitate penetration into the voids of the mortar. The three degrees of dilution were arbitrarily set in the expectation that they would point to the optimum ratio of dilution in field practice. Cubes coated with oil-gasoline mixtures air-dried in about 15 minutes, which was considerably faster than the straight oil-treated specimens.

Coefficient-of-friction tests were conducted on both the untreated cubes and on those that had been oiled. These tests were made on cubes in dry condition, then again while they were wet. There was little difference in friction value between the untreated and the treated specimens, indicating that the light-bodied oil penetrated deeply into the pores. That looked encouraging.

The next step was to test the cubes, to destruction if possible, to determine the relative degree of resistivity to chloride salts afforded by the various oil and oil-gasoline treatments. All test cubes were subjected to routine freezing-and-thawing tests in a 5-ton freezing unit with which one full cycle of freezing and thawing was completed every 24 hours.

Test specimens were placed in metal containers filled with a 10 percent solution of calcium chloride which has a freezing point of approximately 0 F and a higher rate of expansion upon freezing than that of water. The metal containers were set in the freezing tank in which a saturated solution of calcium chloride circulated around the containers and the ammonia pipes. Specimens were frozen and left in the freezer all night at a temperature of approximately −10 F. During the day specimens were thawed by placing the containers in circulating tap water. Each night they were placed back in the freezer for the next cycle. Samples were weighed after each 25 cycles to determine the amount of disintegration as represented by the difference between original weight and remaining weight after the expanding salt brine crystals had carved away some of the mass of the specimen.

Test results

After 100 cycles of freezing of the oil-treated cubes, a solution of the scaling problem became evident. Untreated cubes, both straight portland and natural cement blend, lost 72 percent of their mass which, for practical purposes, represents total destruction.

In contrast, cubes which had been protected with one brush coat of No. 10 oil lost only 4.6 percent of their weight. Losses by specimens protected with two brush coats were 3.6 percent, while loss by specimens having three brush coats fell to the astonishingly low value of 0.3 percent. As might be expected, most of these losses were sustained along the edges of the cubes, the flat surfaces being scarcely affected.

As a result of these tests it was decided that a degree of protection afforded by one brush coat of No. 10 oil would be adequate, since after 25 cycles of freezing the average loss was but 0.5 percent. That meant that, if the pavements could be treated with protective oil coatings every couple of years for the first years of their lives, they might reasonably be expected to be immune to salt action indefinitely.

An important part of the experiment was the determination of the amount of oil actually absorbed by the specimens. Consequently, careful gravimetric checks were made before freezing and thawing tests, and certain equivalents were established to guide in the design of field application methods and equipment. One brush coat of straight oil is equal to a five-minute immersion in a mixture of equal parts of oil and gasoline. This is also equal to an application of 1/20 gal. of 50/50 oil-gasoline mixture per sq yd on horizontal pavement surfaces which

results in the absorption, by the concrete, of 1/40 gal. of straight oil per sq yd of surface.

DEVELOPMENT OF FIELD EQUIPMENT AND PRACTICES

With this information at hand, the problem then became one of translating the results of laboratory experiments into field procedures. Brush coating of miles of highway pavements being unthinkable, it appeared that a truck-mounted spray rig could be devised that would be capable of depositing 50/50 oil-gasoline mixtures on the pavement at the desired rate of 1/20 gal. per sq yd.

A spray bar consisting of a 12-ft length of 1-in. pipe was mounted on the rear of a truck by suitable brackets. Twelve No. 50 Tarrant nozzles were tapped into the spray bar at 1-ft centers. The spray bar was adjusted to a position 20 in. above the pavement so that the natural cones of spray intersected each other approximately 1-in. above the pavement. Thus, one pass of the truck would cover a 12-ft lane. A quick-acting cut-off valve was provided at each nozzle for adjusting the width of sprayed area to various lane widths.

A 500-gal. open tank was mounted on the truck to carry the spraying mixture which was delivered to the spray bar at 35 to 40 psi by a one horsepower gasoline engine driven gear-type pump which could be reversed to refill the tank from 50-gal. drums carried by service trucks. The spraying mixture consisted of equal parts of Stoddard solvent, a mineral spirit used in dry cleaning, and petroleum distillate oil with an approximate SAE rating of 5 and meeting specifications which permit the use of either asphalt or paraffin base oils. Stoddard solvent was adopted to reduce the danger from fire or explosion inherent in gasoline mixtures. The compromise loss of five points in SAE rating of the oil was not considered serious as there is little difference in the viscosity, and it resulted in a considerable saving. Cost of the 50/50 oil-solvent mixture is 11 cents per gal. at the refinery.

Operating at 2 mph, this equipment deposits the protective mixture at the rate of 1/15 gal. per sq yd of pavement. This is 25 percent more than the assumed optimum amount and is considered more than ample to offset the difference in body between the No. 5 oil and the No. 10 oil used in the laboratory. Within fifteen minutes after spraying, the oil is completely absorbed into the concrete, and the solvent has evaporated—permitting resumption of normal traffic. The total cost of one application with this equipment, including operational cost, is approximately one cent per sq yd. Pavements after spraying show no change in color, no feeling of slipperiness, nor any evidence of oil film on the surface. It is evident that the solvent acts to keep the viscosity of the mixture at a low point long enough to secure deep penetration of the oil.

The equipment was developed so late in the summer of 1948 that only a relatively few miles of concrete pavements could be protected before the winter season. However, a total of nearly 60 miles of new concrete highways were sprayed, and these should provide a wide base for field observation of the effectiveness of the oil protective method.

Since results would be difficult to evaluate by reliance on memory and observation alone, the following recording procedure was adopted.

At numerous locations, including those where the application of straight chemicals may be heaviest and most frequent during the winter of 1948, photographs in black and white and in color were made after spraying. In the spring of 1949, and after succeeding winters if desired, these same areas will again be photographed. Direct comparison of the photographs will provide reliable proof of the effectiveness of the treatment. Photographic records include both close-up and oblique views.

OTHER FIELD TESTS

Working in still another direction, the department is now experimenting with admixtures of asphaltic oils in paving concrete mixes in an effort to achieve internal sealing that will resist salt brines. Two asphaltic cutback oils were used: RC-2, a rapid curing oil commonly used for road surface treatment and as a primer for cold patching; and MC-0, a medium curing oil used as a primer on gravel base courses.

A 1000-ft section of experimental pavement was constructed late in the 1948 season. Quantities of oil equal to 2½, 5 and 10 percent of the weight of the cement were added to the concrete mix after the first minute of mixing. Then, the entire batch was mixed for an additional minute.

Present indications are that asphaltic oils, not in excess of 10 percent of the cement by weight, do not seriously affect crushing or tensile strength required in pavement work, cause segregation, or impair workability. However, the 10-percent mix increases the setting time by approximately 25 percent.

Test sections of this same 1000-ft experimental road were also used to check the effectiveness of the oil-solvent protective spray as a curing agent. These sections were sprayed directly after the final finishing operations were completed. At the time of writing this paper, tests on cores taken from the pavement were not completed, nor had the highway been exposed to applications of straight salt. It is expected that freezing-and-thawing tests on the cores may show that better resistance to salt action can be built into the pavement by using asphaltic cutback admixtures. It is also hoped that oil-solvent spray will prove to be an effective curing agent imparting such benefits at the time the pavement is constructed as well as providing resistance to salt action later.

There is one serious hurdle standing in the way of protecting all existing concrete pavements with the oil-solvent mixtures. It has been found that pavements cured with asphaltic curing agents should not be sprayed because the solvent combines with the residual asphalt to make a slippery mass that does not dry readily. A dangerous, greasy surface film is created. Fortunately, this method of curing has not been used recently, so the oil-solvent spray can be utilized to protect our recently constructed concrete highways.

CONCLUSIONS

Normally, research in the evolution of new methods follows a procedure of first developing a theory and then conducting experiments to either prove or disprove it before making actual field tests. In this instance, the approach to the problem of combatting salt action took on a reverse form. Having practically stumbled on what it is hoped will prove to be a rational solution, it was tested in the laboratory and is being tried in the field. Conclusions to date suggest the following theory.

A small part of the mass of concrete is made up of tiny, interconnected voids or cells normally filled with air, or by water if present. These voids are too small to readily accept or convey particles of paint or other heavy bodied liquids because of their high viscosity and high surface tension. That is why paint merely coats the surface of pavements when applied and soon wears off under traffic.

On the other hand, consider a single void just under the surface of a concrete pavement recently sprayed with an oil-solvent mixture. The fluid spreads into a film 0.01 in. thick over the entire surface. It percolates into subsurface connecting voids along the lacework of minute connecting passageways. As the fluid spreads downward, the solvent evaporates upward leaving an oil residue which either fills the myriads of tiny voids or forms a coating on the walls. Thus, a defense in depth has been established against attack by salt solution.

Returning to this typical section some months later, we find the pavement surface above it coated with a solution of salt brine deposited after an application of salt to an ice coating. However, the brine cannot enter many of the voids because they are oil filled or because much of the capillary action of the material has been destroyed by the oil coating left on the surface of the microscopic openings which previous to treatment literally soaked up the brine solution.

In addition to keeping the brine out of the surface of the concrete pavements, the oil residue, coating the walls of the unfilled pores, tends to reduce the size of the water particle and, consequently, its expansion upon freezing and resultant shearing stress. Also, by coating the voids with oil, the deterioration caused by direct chemical action of alkaline solutions would appear to be reduced.

Increasing axle loads and use of chemicals for ice control require of concrete pavement more strength and durability. With the opposing effects of air entrainment on these properties in mind Mr. Noble discusses New Jersey experiences with air-entraining concrete.

Experience with Air-Entraining Concrete in New Jersey*

By CHARLES M. NOBLE†

SYNOPSIS

This paper outlines experience in New Jersey with air-entraining concrete utilized primarily to combat attack by de-icing chemicals. Damage to concrete pavements in New Jersey usually is associated with heavy loads. Freedom from disintegration troubles, except in isolated cases, is attributed to excellent aggregates, rigid laboratory control, mix design, inspection of material and plant equipment and construction supervision at the job site.

Air-entraining portland cement concrete was first specified for a concrete pavement contract in 1945 and since then has been used on many contracts with notable success to prevent deterioration due to scaling. The same high standards for materials, mixture design, laboratory control, field inspection and attention to details must be maintained as with ordinary concrete. Pavements, thus far constructed in New Jersey with air entrainment have not scaled or shown signs of disintegration when ice control chemicals have been properly mixed with abrasives at a rate not exceeding 75 lb per cu yd of abrasive. It is too early to form judgment but indications are the results achieved fully justify the use of air-entraining concrete.

INTRODUCTION

The production of structurally sound portland cement concrete has not been a problem in New Jersey and this paper will outline experience with air-entraining concrete utilized primarily to combat attack by de-icing chemicals. Damage to concrete pavements on the State Highway System is associated primarily with heavy loads and this is understandable when it is realized that for some years tire sizes have been manufactured which legally permit axle loads of 34,400 lb. Tire sizes now manufactured permit axle loads in excess of 40,000 lb. Axle loads of 40,000 lb have been weighed, with a gross of 98,000 lb.

*Presented at the ACI 45th annual convention, New York, New York, February 23, 1949. Title No. 45-29 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE V. 20, No. 7, Mar. 1949, *Proceedings* V. 45. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than July 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

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Disintegration of portland cement concrete is not and has not been a problem to the New Jersey State Highway Department except in certain isolated cases where dolomite was combined with a particular sand map cracking and the effects of growth indicate incipient disintegration. Freedom from disintegration troubles is attributed to excellent aggregates and aggregate producers who cooperate on gradation and cleanness, rigid laboratory controlled design of mixtures, low water-cement ratio, exacting laboratory inspection of materials at the source and on the job, rigid inspection at the concrete batching or mixing plant, strict inspection of batching and mixing equipment, rigid inspection and construction supervision at the job site where the concrete is placed and the use of Type II cement.[1]

In general, scaling of portland cement concrete pavements has not been a matter of concern to the Department and this is attributed to excellent concrete, care exercised to avoid overworking the surface during finishing operations and avoidance of the use of chemicals for de-icing, except where specifically required for traffic safety and then only when mixed properly with abrasives.[2] Raw chemicals are not applied directly to concrete surfaces by Department maintenance forces.

In recent years, however, due to increased vehicular speeds, greater winter travel, the public demand for clear pavement surfaces regardless of weather conditions and the uncontrolled use of raw chemicals on State Highways by municipalities and bus companies the threat of scaling and resultant progressive disintegration due to chemicals has become of greater concern to the Department.

Fortunately, at this time air-entraining portland cement became commercially available at no extra cost in the New Jersey territory. This paper outlines the history of the use of air-entraining portland cement by the New Jersey State Highway Department and experience with it.

USE OF AIR-ENTRAINING CONCRETE

The first use of air-entraining concrete was in 1945 when air-entrainment was specified to be used in a portion of a concrete pavement contract. The contract involved 55,000 sq yd of reinforced concrete pavement of 10 in. uniform thickness and the use of air-entrainment in 27,000 sq yd was an experiment. The specifications required 3 to 6 percent air content and the cement was furnished with flake Vinsol resin interground in the cement at the mill. At the start of the job difficulty was experienced in obtaining the required amount of air but

[1]A specification for a cement closely approximating the present Type II cement was developed by the late R. B. Gage of the Department in 1932 and specified by the New Jersey State Highway Department long before the adoption in 1941 of the Type II Specification by A.S.T.M.

[2]Maintenance forces are under strict order to use no more than 75 lb of chemical per cu yd of abrasive.

after investigation and consultation between the Department laboratory and the mill it was determined that insufficient Vinsol resin was being interground to induce the required air content. When this was corrected no further trouble developed. This pavement is now going through its fourth winter and although abrasives mixed with chemicals have been applied each winter for ice control, no scaling has been observed.

The next contract, involving 19,000 sq yd of air-entraining reinforced concrete pavement of 10 in. uniform thickness, of a total of 49,000 sq yd, was awarded October 10, 1945. Paving started August 21, 1946 and was completed November 23, 1946. No difficulty was experienced on this job with air-entrainment and no scaling has been observed although chemicals were incorporated in the abrasives used for ice control.

Early in 1946, the question of the continuation of air-entrainment was raised due to the loss in compressive strength brought about by the air-entraining agent. The air-entraining concrete compressive strengths on the first contract ranged from 3249 to 7188 psi, with 31 cores averaging 5718 psi at 28 days. The writer felt that the use of chemicals for ice control was here to stay by public demand and that resistance to chemicals as well as increased durability more than offset the disadvantages of lower strength. Consequently the policy was established of specifying air-entrainment for all concrete pavement. On November 24, 1947, a directive was issued stipulating air entrainment for concrete bridge decks, curbs and railings where splashing of chemicals by vehicles could be anticipated and on April 5, 1948, a directive was issued specifying air entrainment in all concrete except in special and isolated cases where high design stresses were utilized in main load-carrying bridge members not subject to contact with chemicals. To date contracts have been let for 1,018,887 sq yd of air-entraining concrete.

So much for the history of air entrainment in New Jersey. The question is, what do we think of it? We like it. The contractors like it and the contractor's workmen like it. The air-entraining agent produces a more buttery, fatter concrete mix which places and works much more easily than standard mixes. It costs no more and permits earlier finishing, thus saving labor and overtime payments.

Occasionally during periods of high temperature and brisk winds, air-entraining concrete has a tendency to set quickly in the top half-inch leaving the interior in a plastic state. This condition has not proved serious in New Jersey.

Specifications on air entrainment

Present specifications require a minimum air content of 3 percent and a maximum of 6 percent, with the objective of 4½ percent as determined

on the job by an approved air meter. The air-entraining agent may be interground at the cement mill or added at the mixer. If interground, the agent is neutralized Vinsol resin—the quantity is approximately 0.012 percent of the cement by weight. If added at the mixer, Darex AEA in liquid form is utilized in an approximate amount of 0.75 oz per bag of cement; or neutralized (in sodium hydroxide solution) Vinsol resin is added in an approximate amount of 0.012 percent per bag of cement. The cements specified are air-entraining cement: A.S.T.M. Designation C175—Type IIA, except that maximum autoclave is 0.2 percent; and standard cement: A.S.T.M. Designation C150—Type II, except that maximum autoclave is 0.2 percent.

In case the required amount of air is not obtained with air-entraining cement, Darex or neutralized Vinsol resin is added at the mixer but this has been required on one job only during the past two years. In general, contractors utilize air-entraining cement rather than add the agent at the mixer.

Air bulking and concrete curing

To minimize the reduction in strength due to the air-entraining agent, the bulking caused by the air should be taken into account. For example, in the summer of 1947, the cement content on a paving job consistently ran below requirements at the end of each day's run in spite of precise laboratory control of the mix and at the mixer. No allowance had been made for the air content. For ordinary cement, the specified method of designing the proportions of cement, fine and coarse aggregates is based on the void content of the coarse aggregate. The design requires enough fine aggregate to fill the voids and allows approximately 10 percent excess. The Department believes this results in stronger and more durable concrete than an over-sanded mix. The air content causes a bulking or increased yield and it is necessary to reduce the quantities of the coarse and fine aggregates by the amount of this bulking to retain the same cement content as specified for standard mixes. It is now standard procedure for the Department to adjust the mix to allow for air bulking. This has reduced the loss in strength. In addition it has been found that the strength loss is only temporary, as much as 25 percent being regained the first year and it is anticipated that the entire loss will be regained in a few years. However, the lower strength immediately after placement may tend to promote early transverse cracking in concrete pavements, especially in relatively long slabs. For this reason, air-entraining concrete in conjunction with relatively long slabs may warrant the use of a curing method that provides sufficient insulation to prevent excessive changes in slab temperature during very early life. This opinion is based on the observed early transverse cracking,

at close intervals, of slabs 56 ft and more in length constructed of air-entraining concrete. In these instances the cracking occurred exclusively in those slabs that were constructed during the morning of hot, sunny days followed by relatively cool nights, and which had undergone appreciable temperature changes and resulting volume changes, within a matter of hours after construction. As a result of investigations made to determine the cause of this cracking the Department feels that insulation to minimize slab temperature differentials during the first 24 hours is at least as important and possibly more important than maintaining an adequate moisture content for purposes of proper hydration.

Control of air content

The secret of success with air-entraining concrete is the air content and this requires a laboratory representative on every job to make air-content determinations with an approved air meter and making adjustments when the air content is not as specified. On a recent job involving 8000 sq yd of pavement, 84 field tests for air content were made. Air determinations are made in accordance with the procedures outlined by H. W. Russell, Engineer of Materials, Illinois Division of Highways, Springfiled, Illinois, in his paper entitled "Measurement of Air Content of Concrete by the Pressure Method," presented before the A.S.T.M. in 1947.

SUMMARY

It is concluded that air-entraining concrete is a step forward in the production of sound, durable concrete but it must not be considered a cure-all justifying letting down the bars in specifications, laboratory control and field inspection. The same high standards for materials, mixture design, laboratory control, field inspection and attention to details must be maintained as with ordinary concrete. It is not believed at this time that the use of air-entrainment justifies the application of raw chemicals to concrete surfaces for ice control. It is believed that chemicals should always be mixed with abrasives in the smallest percentage that will consistently safeguard the public during icy weather. Pavements thus far constructed in New Jersey with air-entrainment have not scaled or shown any signs of disintegration when ice control chemicals have been properly mixed with abrasives at a rate not to exceed 75 lb per cu yd of abrasive. It is too early to form a judgment of the effect of air entrainment on long range durability of cement

concrete pavements and structures but this Department believes the results achieved thus far fully justify the use of air-entraining concrete.

ACKNOWLEDGMENT

The New Jersey State Highway Department is indebted to the late R. B. Gage and his successor, Fred H. Baumann, for the excellent record in producing sound and durable concrete during the past thirty years.

Grateful acknowledgment is made to Department Staff members Fred H. Baumann, Alex. W. Muir, Harry D. Robbins and William Van Breemen for their assistance and comments in the preparation of this paper.

From architect's concept to durable finished structure is a difficult transition if proper consideration is not given to the limitations of materials and construction technique. Mr. Hogan emphasizes the elements of design necessary to achieve the desired architectural effects while at the same time conforming to good construction practice.

Design Details for Architectural Concrete*

By J. J. HOGAN†

SYNOPSIS

"To achieve beauty with concrete, as with any other material, requires studied design and skillful craftsmanship" are the words of architect Robert D. Murray. This paper directs attention to the matter of "studied design". The skilled craftsmanship required to construct architectural concrete must be preceded by proper design by the architect and engineer. The designer must understand and appreciate the fundamental requirements of this type of construction. This paper discusses considerations appropriate to achieving the desired texture of surface as affected by form materials selected. Ornamentation should be considered in every detail from the standpoint of practicability. Construction joints are inevitable and should be specified in logical locations. Similarly control joints should be specified with an understanding of where the most likely cracks due to volume change will occur and the design of reinforcement should be coordinated with the location of the control joints. Numerous illustrations support the text.

INTRODUCTION

Papers have been presented before the Institute on the subjects of mixing, placing and curing of architectural concrete and on the construction of forms for architectural concrete structures. As a result of the dissemination of such information through the Journal of the Institute and by other agencies a marked improvement in job craftsmanship is evident but there is still a definite need for further discussion of these subjects. Old and particularly bad habits are not changed readily, lessons are not learned quickly. There is another phase of architectural concrete which has been given even less attention than the construction technique and very much less than it deserves. In the words of architect Robert D. Murray, "To achieve beauty with concrete, as with any other material, requires studied design and skillful craftsmanship". It is to the matter of "studied design" that attention is directed here.

*Presented at the ACI annual convention, New York, N. Y., February 25, 1949. Title No. 45-30 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 7, Mar. 1949, *Proceedings* V. 45. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than July 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

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DESIGN REQUIREMENTS

Before skillful craftsmanship can play its part, the design executed by the architect and engineer may do much to spell success or failure in achieving beauty and perfection in the completed structure. The quality of construction is dependent to a great extent on what is incorporated in the plans and specifications. The finest design from the standpoint of line, mass, balance, proportion and the other factors considered by judges in a competition is not sufficient. The completed structure may be disappointing unless the designer understands and appreciates the fundamental requirements of this type of construction.

Texture

After an architect has conceived the design of an architectural concrete building he must determine the surface texture which will be in keeping with the design in order to translate his mental picture into a satisfying reality. He may use boards ranging in nominal width from 3 in. to 10 or 12 in. They may be dressed and matched or square-edged and rough. Or he may elect to use plywood or other material manufactured in large panels. Whatever form sheathing material he selects it will leave its everlasting imprint on the structure so he must bear in mind the characteristics of the chosen material. For example, boards up to 8 in. wide will produce a satisfactory surface if nominal 1-in. material is used but with boards wider than 8 in. undesirable cupping will result unless 2-in. thick boards are used. Such heavy sheathing means more costly forms so there may be an inclination to specify a minimum spacing of studs wider than will give good results even though the boards have sufficient strength to resist the pressure of the concrete. Two-inch sheathing should be nailed not farther apart than 24 in. to prevent twisting. If the architect has disregarded these facts in writing his specifications he will be disappointed in the final result.

The architect should also realize that the form lumber obtainable today is not kiln-dried or at best inadequately dried. If he specifies wide sheathing boards he should anticipate greater shrinkage than with narrower boards which will tend to give more pronounced joint lines due to the fins along slightly open joints.

Just as an architect takes into account brick sizes in laying out story heights and sizes and locations of openings, he must also consider the actual widths of the sheathing material for a concrete job. Economy will depend upon minimizing the cutting of lumber and consequent waste and a pleasing design may be ruined by an unpremeditated patchwork effect, resulting from trying to accomodate the sheathing material to building dimensions not properly determined. It is particularly important where plywood or other large panels are used that the standard

dimensions, usually 4 x 8 ft, be borne in mind. Because of the larger module the use of random sizes here and there is even more objectionable than for narrow boards and the waste due to cutting is particularly expensive.

Ornament

At the time the architect is considering the influence of the form sheathing on the design he should take into account any ornamentation involved. Every detail should be considered from the standpoint of practicability. Sometimes it is unfortunate that concrete is such an adaptable material. Its very adaptability may lead to expensive and unsightly abuses. Restrained ornamentation that can be executed with simple wood or plaster molds will generally meet most design requirements. Such ornamentation is economical to form and is conducive to best results in the final product.

Incised ornament, that is to say, any ornament that is flush with or recessed back of the wall surface is generally more economical than projecting detail. The latter requires cutting the main wall forms and the construction of additional forms which are often difficult to hold securely against leakage. For incised ornament, on the other hand, the negative wood moldings or even plaster molds can be applied to the face form after it is erected without cutting or waste.

Very fine detail, extremely thin members and deep undercuts should generally be avoided because of possible breakage, greater cost and the fact that such ornament is not fundamentally in keeping with the nature of the material. Bold, strong, detail with full rounded forms such as the fluting shown in Fig. 1 is conducive to perfection in execution and to least cost. Details of form construction are shown in Fig. 2. It should not be misunderstood that intricate detail cannot be executed. If the architect's conception of a building calls for a traditional Greek design in the Corinthian order, even the capitals with their delicate undercuts can be formed successfully with piece molds which are simply plaster waste molds made in small sections. More labor is required in the erection of such forms and greater care must be exercised in stripping at consequently greater cost. Effective results can usually be obtained with details that can be formed with simple wood moldings and even with stock moldings or combinations of them. Substantial savings are made when architects keep this in mind.

Construction Joints

The technique of construction should always be taken into consideration in the design of any building regardless of the building material. Construction joints are an inevitable part of every architectural concrete building. There are logical locations for such joints and the

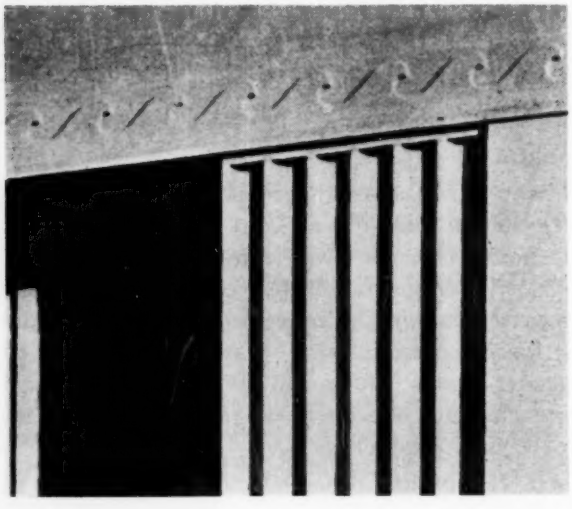

Fig. 1—Full rounded fluting and bold detail permit easy form building and perfect execution at minimum cost

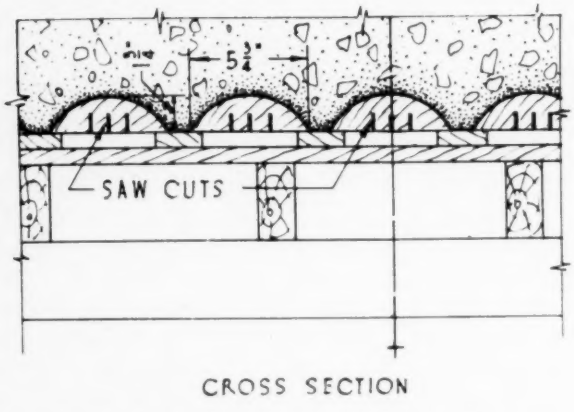

Fig. 2—Form details for fluting shown in Fig. 1

designer should recognize and accept that fact. Properly made, construction joints need not be unsightly but it is advisable to locate them where least noticeable. Frequently some architectural detail can be used to obscure a joint. High lifts should be avoided as the difficulties of placing in thin walls multiply rapidly as the height increases. Reasonable lifts facilitate placing, make it easier to avoid segregation and water gain and result in better surfaces. A reasonable rate of placing the concrete should be considered when locating joints. Generally, a rate of about 2 ft per hour is most conducive to good workmanship and lifts of 6 to 8 ft between joints are desirable and 12 ft should be the absolute maximum.

Since sufficient time must be allowed for the concrete between windows or other openings to settle and shrink before the concrete above is

placed, the tops of such openings are "must" locations for construction joints. Similarly, it is desirable to locate a joint at the sill line. By so doing the joints are broken up into short lengths by the openings, making them relatively inconspicuous. Moreover, cracking at corners of openings is thereby minimized.

Having decided upon the proper location of construction joints, the designer should show them on his drawings and no deviations from those locations should be permitted without approval. Fig. 3 shows a completed building while Fig. 4 is a drawing of the front elevation on which the construction joints are indicated. It will be noted that there are no high lifts, the joints are as short as possible and are located at natural lines in the design. The joint below the first-story windows is at the top of the water table where there is a slight reveal which makes the long uninterrupted joint at that level quite inconspicuous.

Fig. 3—Construction joints may be obscured by architectural detail as in the Borden Creamery building, San Antonio, Texas.

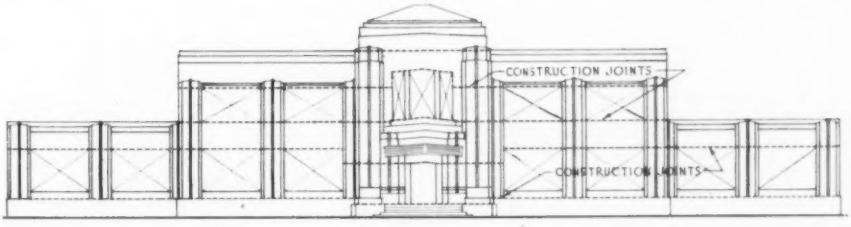

Fig. 4—Locations of construction joints in building shown in Fig. 3

Vertical construction joints are sometimes necessary. They should be located at re-entrant angles or at the edge of a pilaster or some decorative feature may be provided to obscure such joints.

Control Joints

In addition to construction joints it is necessary in every architectural concrete building to provide contraction joints, commonly called control joints. There is always some restraint against free movement in a building so that stresses are set up which must be taken into account. It is impractical, if not impossible, to provide sufficient reinforcement to prevent cracking so control joints must be provided at suitable intervals and locations to pre-determine the location of cracks which will relieve the stresses produced by volumetric changes.

The architect should show the location of control joints on his drawings as in Fig. 5. In walls having openings at frequent intervals joints should be spaced not more than 20 ft apart and somewhat closer than this below the first story or basement windows down to the footings. The closer spacing in the lower part of the wall is necessary because the foundation is subject to little volume change while the wall above is exposed to variations of temperature and moisture and there is no chance for relief due to deflection of columns, mullions or piers. In walls without openings joints should not be more than about 25 ft apart.

In general, control joints should be placed at wall openings because the wall is already weakened at such locations. Where single small openings occur at more than 20-ft intervals there should be a joint in line with each jamb below the openings at first-story level because it is there that cracks are most likely to appear. Above the first-story openings a single joint on the centerline of each opening will generally suffice, but joints on the jamb lines are preferable if shear in the spandrel will permit. Joints should also be provided in the solid wall between

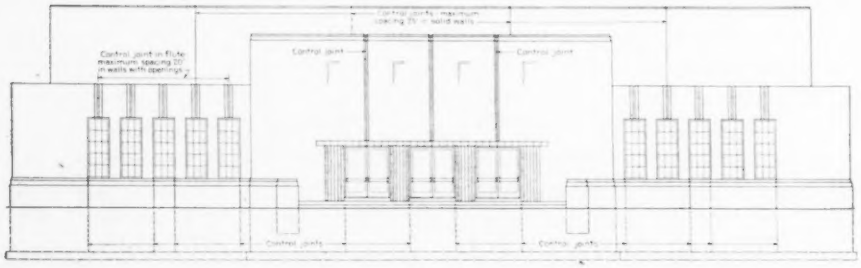

Fig. 5—Control joints located with regard to structural requirements and architectural effect

openings if necessary to maintain a maximum spacing of not more than 25 ft.

Steel columns embedded in a concrete wall weaken the wall section more than openings. Joints should be placed in the planes of such columns and at intermediate sections in line with openings if the columns are more than 25 ft apart. Specifications should require that the column be painted with an asphaltic paint to break the bond and no mesh or other reinforcement should pass through the control joint. If there is a pilaster at the steel column, which is often the case, a detail such as that in Fig. 6 should be included in the drawings to show that the wall steel is to be carried well up to the web of the column and the bars should be hooked in a vertical plane. This is done to insure against cracks at the edges of the pilaster.

A detail of the type of control joint desired should be shown in the architect's drawings. Some architects are using control joints as decorative features and make them quite wide but the general practice is to make them as narrow as possible. The joints are usually formed with wood or metal strips which leave narrow grooves opposite each other in the two faces of the wall to reduce the wall section at least 20 percent. Typical joint details are shown in Fig. 7 and 8. At least half of the horizontal wall reinforcement should be cut at the joints and any extra reinforcement provided at the heads and sills of windows and at tops of parapets must be stopped off or cut at control joints except where needed for structural strength.

The architect should show how he wants control joints made where they cut across window sills and copings. Sometimes when this detail

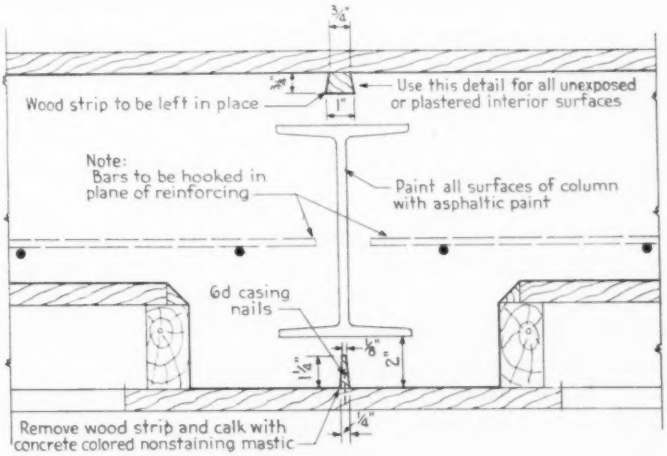
3/4"
Wood strip to be left in place
1"
Use this detail for all unexposed or plastered interior surfaces
Note:
Bars to be hooked in plane of reinforcing
Paint all surfaces of column with asphaltic paint
6d casing nails
1/8"
2"
1 1/2"
Remove wood strip and calk with concrete colored nonstaining mastic
1/4"

Fig. 6—Form assembly for pilaster at steel column

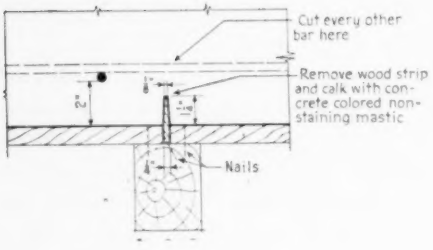

Fig. 7—Form assembly using wood control joint strip

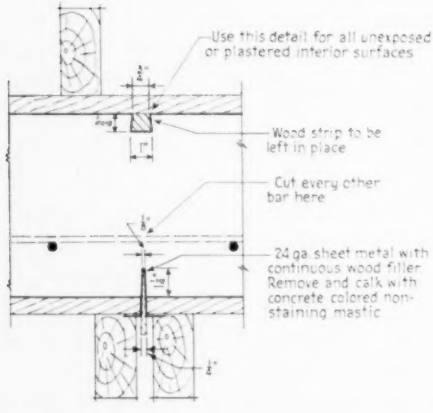
Use this detail for all unexposed or plastered interior surfaces
Wood strip to be left in place
Cut every other bar here
24 ga. sheet metal with continuous wood filler Remove and calk with concrete colored non-staining mastic

Fig. 8—Form assembly using removable metal control joint strip

has been omitted the joints have also been omitted with the inevitable result that an irregular crack has occurred in line with the joint below. At such locations a joint may be tooled across the sill or parapet in the plane of the control joint and the groove may be filled with caulking compound. Better still a detail such as shown in Fig. 9 may be used. The metal joint cover is embedded in the fresh concrete which will hold it securely in place when hard. The joint cover causes the crack to occur where desired, covers it and seals it against leakage.

Reinforcement

Although, as already mentioned, it is impractical to provide sufficient reinforcement to prevent cracking in a long wall and for that reason control joints are used, it is necessary and practical to provide adequate reinforcement between joints when properly spaced to resist volume change stresses. To be most effective small bars with a type of deformation to give maximum bond should be placed relatively close together rather than using larger bars at wider spacings. Generally $\frac{3}{8}$-in bars are preferable to larger sizes and the horizontal reinforcement should be at least equal to 0.25 percent of the wall area. In the vertical direction 0.15 percent reinforcement is sufficient. The size and spacing of bars recommended for walls of various thicknesses are shown in Table 1.

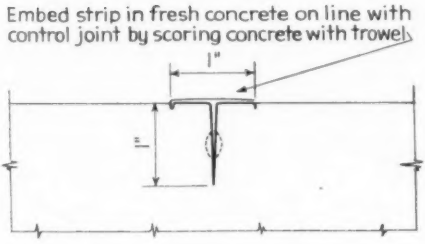

Fig. 9—Lead-coated copper joint strip

TABLE 1—RECOMMENDED SIZE AND SPACING OF REINFORCING BARS IN CONCRETE WALLS

Wall thickness in.	Horizontal reinforcement	Vertical reinforcement
6	⅜-in. ϕ @ 8-in. in outside face of wall	⅜-in. ϕ @ 8-in. in outside face of wall
8	⅜-in. ϕ @ 6-in. in outside face of wall	⅜-in. ϕ @ 8-in. in outside face of wall
10	⅜-in. ϕ @ 10-in. in both faces of wall	⅜-in. ϕ @ 12-in. in both faces of wall
12	⅜-in. ϕ @ 8-in. in both faces of wall	⅜-in. ϕ @ 12-in. in both faces of wall

Specifications should require that the bars be tied at each intersection with a non-slip tie and where two curtains of steel are used it is advisable to include a detail in the drawings such as Fig. 10 to show how the curtains are to be fastened together with tie spreaders at frequent intervals to make a rigid web of reinforcement.

Additional reinforcement is often necessary around wall openings to take care of concentrations of stress. Where control joints are on the centerline of an opening extra reinforcement should be provided at the sides and above and below the opening, as shown in Fig. 11(A), these bars should extend at least 2 ft beyond the corners of the opening. Where there are control joints on the jamb lines below the opening, shown in Fig. 11(B), the extra bars may be omitted from the sill and the vertical bars at the sides need extend only 2 ft below the head of the opening. Where the control joints are on the jamb lines above and below the opening no extra reinforcing is required unless the dimensions of the lintel are such as to require it (Fig. 11(C)).

Difficulty is sometimes experienced in placing concrete in walls because of a congestion of reinforcing bars and the designer should avoid this condition insofar as possible. For example, where there is a beam at the floor line carrying a high percent of steel, the beam should be

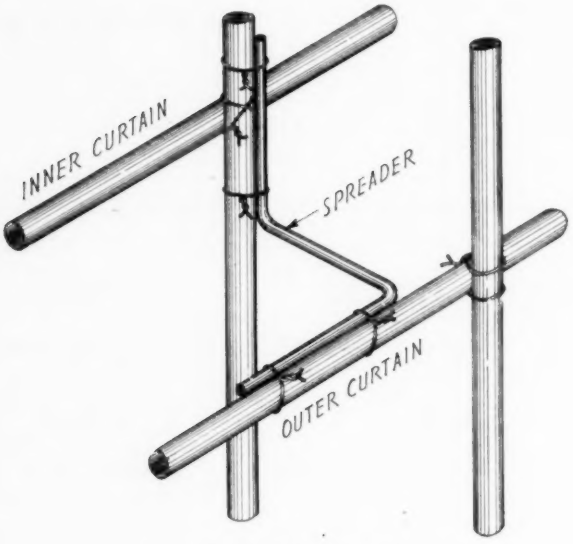

Fig. 10—Tie spreader

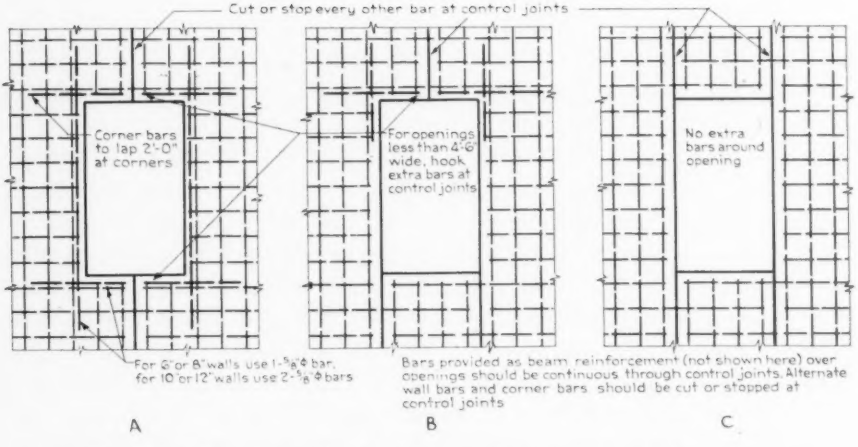
Cut or stop every other bar at control joints
Corner bars to lap 2'-0" at corners
For openings less than 4'-0" wide, hook extra bars at control joints
No extra bars around opening
For 6" or 8" walls use 1-⅝"ϕ bar, for 10" or 12" walls use 2-⅝"ϕ bars
Bars provided as beam reinforcement (not shown here) over openings should be continuous through control joints. Alternate wall bars and corner bars should be cut or stopped at control joints
A
B
C

Fig. 11—Alternative methods of placing reinforcement around openings

made wide enough so that the steel does not interfere with placing concrete in the wall. Where there are wide canopies there is sometimes so much steel carried from the slab into the wall as to make placing difficult. It is often desirable to thicken the wall at such locations, both to make placing easier and to be sure to get complete and adequate coverage over the steel.

Stairs parallel to and adjoining a wall should be designed independent of the wall. If this is not done, and the stairs are framed into the wall, both the forming and the construction of the wall are greatly complicated. Bars may be left projecting from the wall to tie the stairs, but it is inadvisable to provide a recess in the wall on account of the tendency to produce cracking.

Wherever stirrups are required in spandrel beams, there should be support bars at the free ends of the stirrups. Often the plans do not indicate these bars nor do the specifications specifically require them; hence, they are not installed by the steel erector. They should always be shown on the plans to make certain they will be used.

Because of the concentration of stress in parapets, more reinforcement should be used above the roof line than in the wall below. Fig. 12 shows the recommended minimum reinforcement for parapets of various thicknesses. Reinforcement for copings is also indicated. As in con-

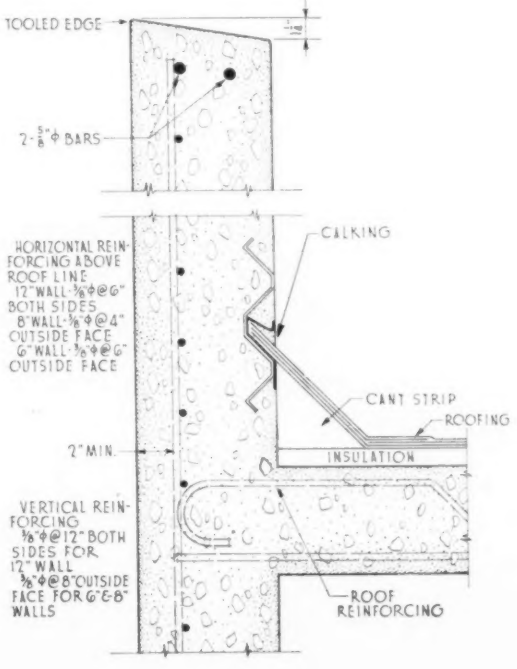

Fig. 12 — Recommended minimum reinforcement for parapets

struction with other materials, a ragglet should be provided in the back of the parapet to receive the roof flashing. In areas where there is considerable snow, it is desirable to coat the entire inside surface of parapets with roofers pitch.

CONCLUSION

The importance of practicability in design of architectural concrete buildings cannot be too strongly emphasized. Details should be chosen which make it reasonable to expect the contractor to execute them in a workmanlike manner. What is expected of the contractor should be clearly shown on the plans and called for in the specifications. Such practice is conducive of good results.

Sound design in architectural concrete must be followed by skillful execution by the contractor to assure satisfactory results. Mr. Oberly points out common difficulties encountered on the job and discusses techniques which have been used successfully to overcome them.

Construction Practices for Architectural Concrete*

By E. B. OBERLY†

SYNOPSIS

Detailed plans and carefully drawn specifications executed by a contractor and superintendent with the know-how of form construction are necessary for good architectural concrete. This paper describes some of the more important construction practices to be considered to achieve the results desired. Form details and concrete quality are discussed in considerable detail, the former being supported by illustrations.

INTRODUCTION

Appearance is of paramount importance to the success of architectural concrete, therefore, every operation in the construction must be planned and executed with this in mind. The architect has a right to demand and expect finished concrete surfaces of uniform texture, fairly uniform color, sharp detail and true arrises. His specifications for form construction, for concrete ingredients and for placing and finishing the concrete must be more comprehensive than those generally used for ordinary structural concrete jobs. Regardless of how detailed he may make the specifications, however, it is the contractor and his superintendent who must have the "know how" to detail and construct the forms and be on constant watch during all operations to see that correct procedures are followed.

Some of the more salient features of form construction have been discussed in other papers, but experience since they were published has developed some modifications in details. Likewise, the requirements as to concrete for architectural concrete structures have been described but additional stress on certain ones appears to be warranted.

*Presented at the ACI 45th annual convention, New York, N. Y., Feb. 24, 1949. Title No. 45-31 is a part of the copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE V. 20, No. 7, March 1949, *Proceedings* V. 45. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than July 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

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FORM DETAILS

Prevention of leakage

Form design and construction must be approached with a different viewpoint than that for ordinary structural concrete. In most of the latter work, some leakage and irregularities in formwork may be tolerated, so long as the forms are strong enough to carry the weight of the concrete and the imposed loads and are built fairly true to line and grade. In architectural concrete, on the other hand, there must be no leakage through the forms and bulging and other irregularities must be so inconsequential as not to detract from the appearance of the finished surfaces.

Every conceivable precaution must be taken to prevent leakage through the forms. Chamfer strips at corners and edges are helpful for this purpose. Wherever possible, the forms should be designed so that pressure of the fresh concrete will tighten joints rather than tend to loosen them. Fig. 1 is a good example of how this can be done and at the same time make it easier to build the forms. It shows a section through a typical pilaster. Such a form should be made on the bench, in one piece if not too large, or in two or three pieces if it is a large pilaster. Note that the block between the pilaster wale and the main wale is cut slightly loose. As pressure develops in the wall, the main wale is pushed outward. With slight movement it bears against the block, but the movement tightens the joints at A and B. Note also that the sheathing is so arranged that end grain marking occurs on the reveal and not on the face of the wall, thus making the trace of the end grain less conspicuous.

Form bulging

A common source of trouble is bulging of forms at external corners, resulting in both poor alignment and leakage with subsequent ragged corners, sometimes to the extent of exposing the aggregate at the corners. This point has been stressed in most literature on architectural concrete

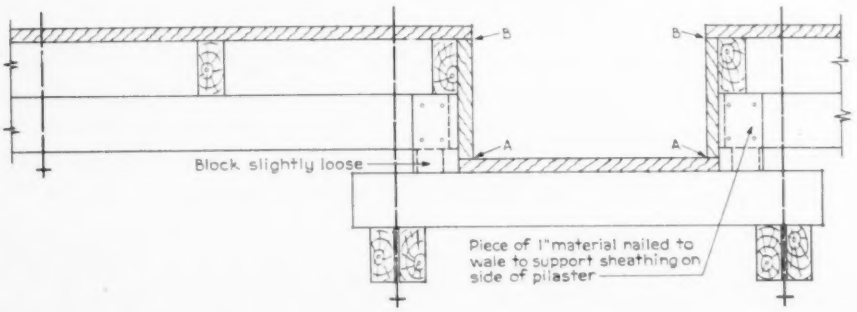

Fig. 1—Form designed so that pressure of fresh concrete tightens it to prevent leakage.

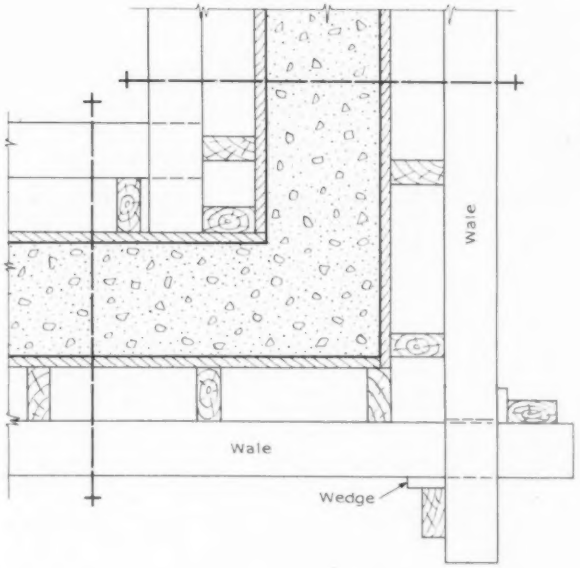

Fig. 2—Form construction to provide positive corner locking

construction, yet it is surprising how often it is overlooked and no provision made for locking the form corners.

Fig. 2 illustrates one method of constructing the forms to provide locking of the corner. The wales are extended beyond the intersection far enough to permit two vertical strips to be nailed in the corners. Wedges are driven between these strips and the wales to tighten the corner. The wedges should be re-driven tight just before placing the concrete to take up any shrinkage that has occurred in the wood. As in all cases, this wedging must be done before starting to place the concrete, as leaky forms cannot be corrected while placing is in progress. An interesting detail in this corner assembly is that a stud covers the joint in the overlapping sheathing, which is of further assistance in preventing leakage at this point.

Fig. 3 shows another good detail for locking the form corners. In this case a tie is used instead of the wood strips and wedges. Since there are no wedges which can work loose, a set of wales can be placed and tightened on one level around the building without much chance of subsequent loosening when tightening the wales above or below.

Plywood form sheathing

Smooth surfaces required on much of our present-day work often are obtained with plywood form sheathing. This should be of structural grade made especially for form use and preferably should be oiled at the factory. If oiled on the job, sufficient time should be allowed for the oil to penetrate and any surplus oil should then be wiped off before

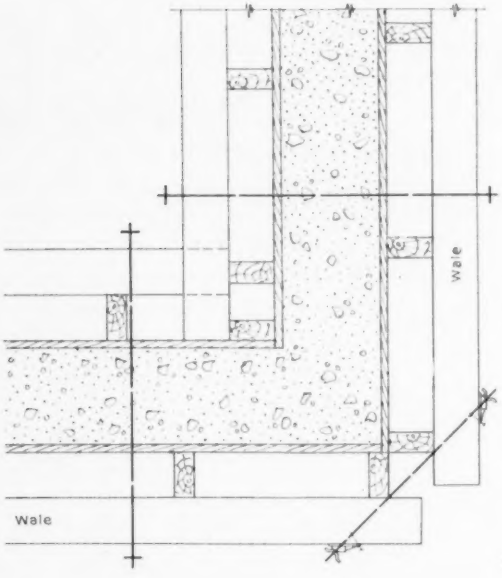

Fig. 3—Form locked by metal ties

the plywood is used. Fig. 4 illustrates about what is standard practice in the use of structural plywood. Plywood $\frac{5}{8}$ or $\frac{3}{4}$ in. thick can be used directly against studs without backing when the outer plies are at right angles to the studding. Structural plywood usually has five plies with the fibers in the two outer and middle plies in one direction and the fibers of the other two plies in the opposite direction. Thus the plywood is stronger and stiffer in one direction than the other. In spite of this, contractors on some jobs have persisted in turning the outer plies parallel to the studding. The result may not be serious on the first use of the plywood and under average placing speed, but with faster placing the deflection may be pronounced. With the second use of the plywood there will be noticeable deflection and with the third and additional uses, a deflection of $\frac{1}{8}$ to $\frac{1}{4}$ in. can be expected with $\frac{3}{4}$-in. plywood on studs placed on 16-in. centers. Since there is no saving in cost and objectionable deflection will occur, the practice should not be allowed.

Ornamentation

Belt courses, cornices or other ornamentation where continuous members occur may be formed with wood moldings cut to the proper shape. While some contractors may prefer a waste mold for this particular detail, the wood mold has the advantage of being more easily held in alignment because the various members are broken at different points, while in a plaster waste mold the entire form would be cut in two at one point. Soft-grained wood that does not warp or split easily should be used. Soft

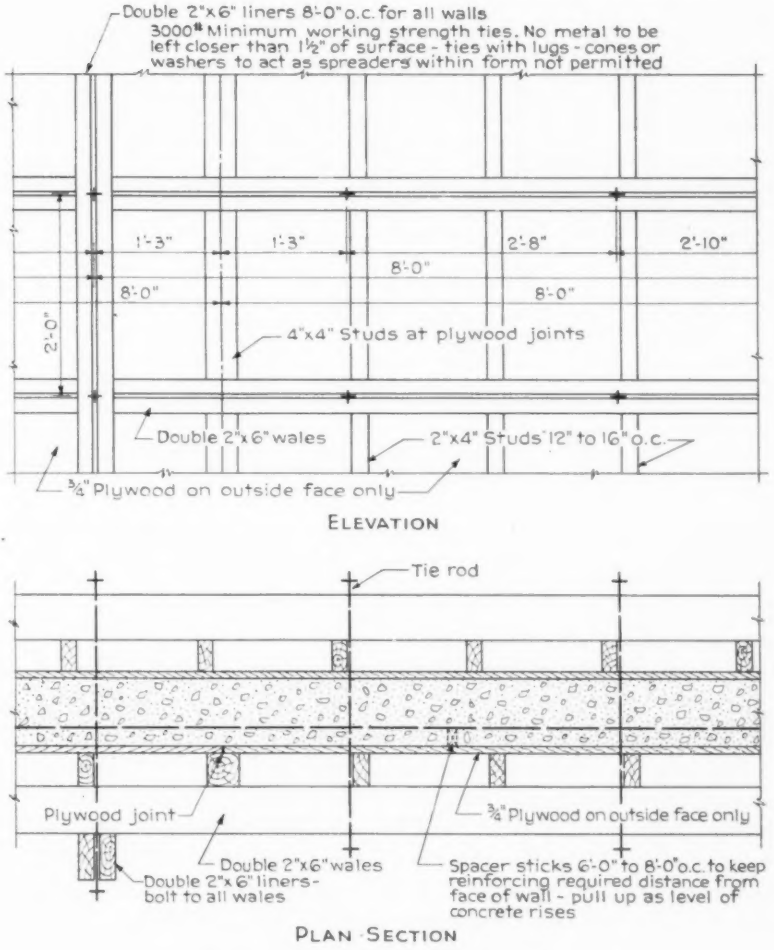
Double 2"x6" liners 8'-0" o.c. for all walls
3000# Minimum working strength ties. No metal to be left closer than 1½" of surface - ties with lugs - cones or washers to act as spreaders within form not permitted
1'-3"
1'-3"
8'-0"
2'-8"
2'-10"
8'-0"
8'-0"
2'-0"
4"x4" Studs at plywood joints
Double 2"x6" wales
2"x4" Studs 12" to 16" o.c.
¾" Plywood on outside face only
ELEVATION
Tie rod
Plywood joint
¾" Plywood on outside face only
Double 2"x6" wales
Double 2"x 6" liners - bolt to all wales
Spacer sticks 6'-0" to 8'-0" o.c. to keep reinforcing required distance from face of wall - pull up as level of concrete rises
PLAN SECTION

Fig. 4—Form layout for use of structural plywood

white pine is best with soft-grained Douglas fir a second choice. In the detail shown in Fig. 5, the various members are narrow enough so that common stock sizes can be used throughout. Note the generous use of saw kerfs in the back side of members to prevent swelling and warping of the lumber. The lumber should be thoroughly oiled on all sides before building the forms as further precaution against swelling and warping.

Large, heavy wood members should be avoided in detailing for ornamental work as they will swell and in stripping, the corners of the projecting concrete will be fractured. Using several smaller pieces requires less lumber and the members may be arranged so that swelling is away from the concrete, making it easier to strip the form without damage to

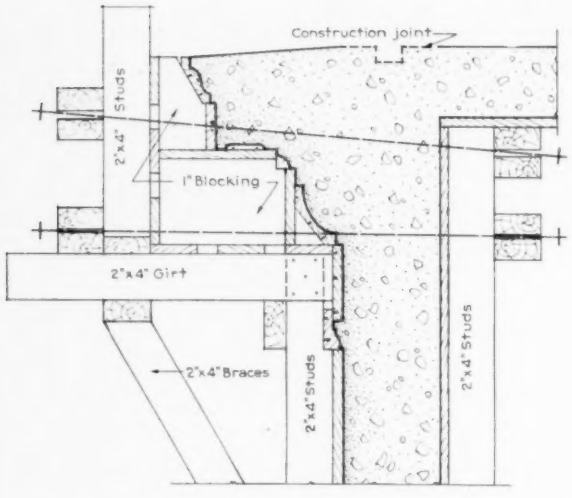

Fig. 5—Wood mold used in cornice form

the concrete corners. This is illustrated in Fig. 6 where the upper detail shows a large member milled from a single wide board in contrast to the smaller pieces shown in the lower detail.

In assembling forms one must keep constantly in mind the steps that can be taken to aid in removing them without injury to the concrete. Boxes, waste molds, wood molds, rustication strips or anything applied to the face of the forms should be nailed lightly so that when the forms are removed, these members will pull loose from the wall form and will remain in the concrete. After these materials have dried out thoroughly and thus have shrunk, they can be removed without much difficulty and without injury to the concrete corners or edges. Some items, such as rustication strips, can be nailed with casing nails long enough so that the points of the nails project through the sheathing. Just before stripping the forms, these nails can be pulled through the strip and sheathing, thus leaving the strip entirely free to remain in the concrete.

As indicated previously, solid strips of wood, even though well oiled, may swell and result in considerable breakage of concrete corners when the strips are removed. Saw kerfs on the back of the member will prevent this trouble by relieving any pressure against the concrete. They should be approximately two-thirds the depth of the member, not more than $1\frac{1}{2}$ in. apart and, in general, there should be one kerf within $\frac{3}{4}$ in. of each edge.

Waste molds are the only solution for some highly ornamental work. They should be made only by experienced ornamental plaster workers who have been given instructions as to how they are to be fitted into the forms. Ordinarily, the waste molds are about 2 in. thick, reinforced

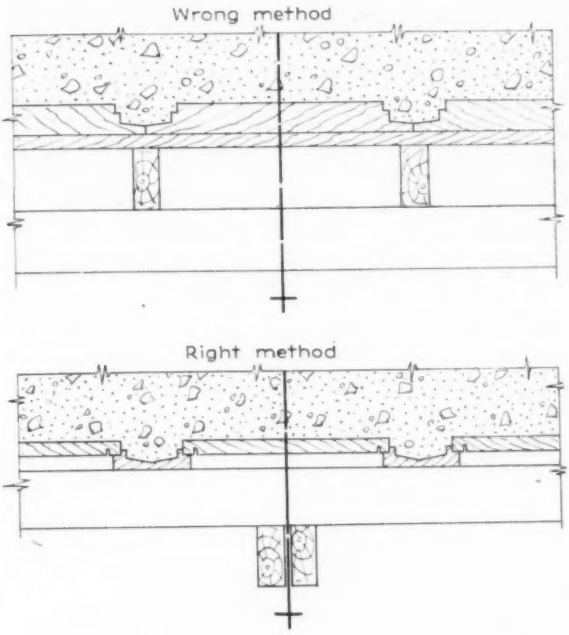

Fig. 6—Proper form construction prevents distortion from swelling and permits easier stripping

on the back with 2x4's which are attached to the mold with burlap dipped in plaster and then wound around the wood. The 2x4's permit easy handling of the mold and are also used to attach the mold to the form.

When waste molds are large and have deep undercuts, it is usually best to wire them back to the studs or wales with enough wires to be sure that all points will be pressed firmly against the form. Any openings between the form and waste mold should be pointed with plaster of Paris or a patching plaster and nailheads driven through the face and around the edge of the waste molds should be countersunk and similarly pointed.

Construction joints

Much progress has been made on all types of concrete work in the production of satisfactory construction joints, both from the standpoint of good appearance and good bond between the successive lifts. On the other hand, some very unsightly joints for which there is no excuse occasionally are made on architectural concrete work. It is just as easy to produce a good joint as a poor one. The designer, of course, should indicate on his plans exactly where such joints are to be located. Rustications or offsets will mask the joints and are advisable where they can be made to fit into the design.

One requirement that is sometimes neglected is to provide means of holding the form tightly against the hardened concrete. As a result, leakage occurs, discoloring the concrete below and the upper lift will project slightly beyond the lower lift. While Fig. 7 was shown in a previous ACI paper, it is repeated here because of its importance in the production of good joints. It illustrates the use of $\frac{5}{8}$-in. bolts cast in the concrete for fastening the form for the next lift. Some types of form ties can be used instead of bolts. The bolt or form tie assembly can be used over and over, as only the nut is left in the concrete if the bolt is used, or the insert if a form tie is used. In re-erecting the forms for the next lift, the contact surface of the form sheathing should not overlap the hardened concrete more than 1 in. More overlapping than this only presents more opportunity for leakage due to irregularities in the wall surface against which the sheathing is to be held.

The construction joint should be only a straight thin line at the surface. If a rustication strip is not used at the joint, a $\frac{3}{4}$-in. wood strip should be nailed to the inside face of the form. The concrete should be placed to a point $\frac{1}{2}$ in. above the bottom of this strip and just before the concrete becomes hard, the top should be lightly tamped to be sure that it is tight up against the strip after it has settled, or shrunk. All surplus concrete and laitance should then be removed.

In resuming the placing of concrete in the new lift, steps should be taken to get good bond and to avoid honeycomb. The hardened concrete should be clean and thoroughly saturated. A 6-in. layer of concrete in which the amount of coarse aggregate is reduced about 50 percent, or a 2-in. layer of cement-sand grout should then be placed on the hardened concrete followed by the regular concrete.

Control joints

It is important, both for appearance and performance, that control joints be carefully installed so they will be perfectly straight and vertical. Care is required during placing of concrete to avoid knocking the joint out of alignment or otherwise injuring it. When control joints are to be calked with a plastic compound, this should be done before the final clean down so that any compound smeared onto the surface is removed. Of course, excessive smearing should be avoided.

Form ties

Various makes of form ties are available for architectural concrete construction as shown in Fig. 8. Important features to look for in selecting form ties are: removal of all metal to a depth of at least $1\frac{1}{2}$ in. from the face of the wall, minimum strength of 3000 lb when fully assembled, adjustable length to permit tightening of forms around all openings and inserts and a design that will leave a hole at the surface not more than

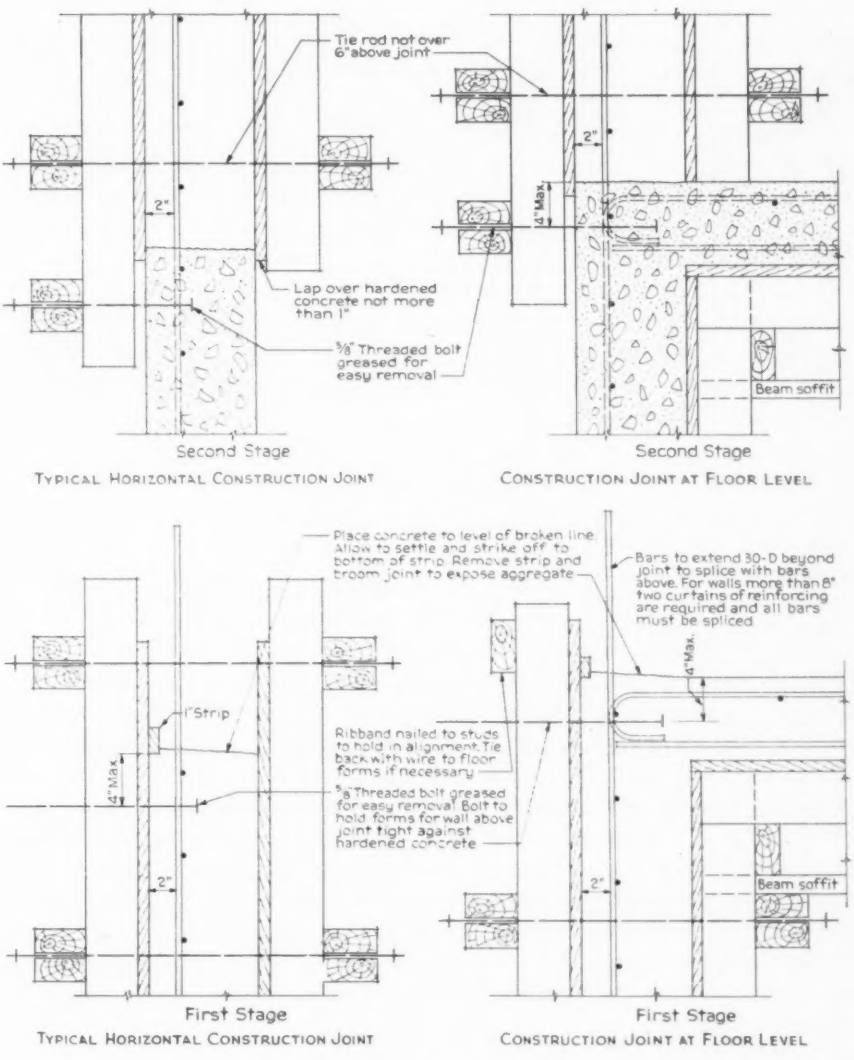
Tie rod not over 6" above joint
2"
Lap over hardened concrete not more than 1"
⅜" Threaded bolt greased for easy removal
Second Stage
TYPICAL HORIZONTAL CONSTRUCTION JOINT
2"
4" Max.
Beam soffit
Second Stage
CONSTRUCTION JOINT AT FLOOR LEVEL
Place concrete to level of broken line. Allow to settle and strike off to bottom of strip. Remove strip and broom joint to expose aggregate
1" Strip
4" Max.
2"
⅜" Threaded bolt greased for easy removal. Bolt to hold forms for wall above joint tight against hardened concrete
First Stage
TYPICAL HORIZONTAL CONSTRUCTION JOINT
Bars to extend 30-D beyond joint to splice with bars above. For walls more than 8" two curtains of reinforcing are required and all bars must be spliced
4" Max.
Ribband nailed to studs to hold in alignment. Tie back with wire to floor forms if necessary
2"
Beam soffit
First Stage
CONSTRUCTION JOINT AT FLOOR LEVEL

Fig. 7—Method of embedding bolts and placing forms for successive lifts

⅞ in. in diameter and not less than ¾ in. deep so that it will hold a patch. Ties should not be fitted with cones or washers acting as spreaders, as tapered and shallow holes cannot be patched properly.

CONCRETE QUALITY AND PLACEMENT

Mix requirements

Architectural concrete demands the use of a plastic, workable mix that can be worked and molded readily into the forms. Surfaces of the hard-

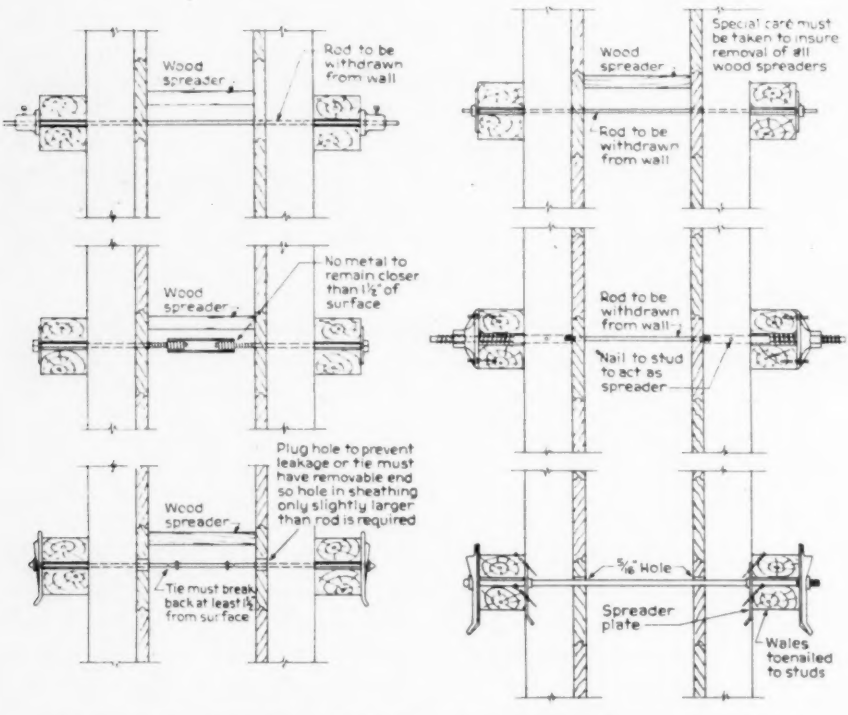
Wood spreader
Rod to be withdrawn from wall
Wood spreader
No metal to remain closer than 1½" of surface
Wood spreader
Plug hole to prevent leakage or tie must have removable end so hole in sheathing only slightly larger than rod is required
Tie must break back at least 1½" from surface
Special care must be taken to insure removal of all wood spreaders
Wood spreader
Rod to be withdrawn from wall
Rod to be withdrawn from wall
Nail to stud to act as spreader
5/16" Hole
Spreader plate
Wales toenailed to studs

Fig. 8—Types of form ties available for use in architectural concrete construction

ened concrete should be free of defects such as sand streaking and honeycomb. It is important, therefore, that sufficient fines be provided in the mixture and for these reasons the fine aggregate should have at least 15 percent passing the 50-mesh sieve and 3 percent passing the 100-mesh sieve and the cement factor should be not less than 5½ sacks per cu yd of concrete. Not more than 6½ gal. of water per sack of cement should be allowed so that the concrete will have the durability required to withstand weathering.

Mixing and placing

On some jobs a mixing plant is set up that is too large for the job and many days or weeks may intervene between successive placings of concrete. It is better practice to select a plant of such size that some concrete can be placed each day, insofar as this is practical. It is more economical and usually will result in better quality concrete as production of concrete is at a slower rate presenting better conditions for placing and consolidating it. With the horizontal and vertical joints which have been found essential for architectural work, there are many logical places to stop placing concrete and it is not necessary to place it contin-

uously in large units. When using mixers too large for the job, there is a tendency to build up layers of the fresh concrete that are too thick. This is apt to result in wavy lines of discoloration on the surface. Layers of fresh concrete should be kept to a maximum of 18 in. and less than this is preferable. While the rate of placing is somewhat dependent on workability of the concrete and size and shape of the section, in general it should be held under 2 ft per hour. Excessive pressures are then avoided, the tendency for water to rise to the surface is minimized and the placing crew will have the time necessary for proper consolidation. The concrete should be spaded or vibrated into all corners and along form surfaces, but even if vibrated, some spading is necessary in the corners and angles as vibrators will not fill these places at all times. On many jobs small electric hammers moved along the forms at the point of placement have been found helpful. Vibration should be sufficient only for complete compaction, not to the point where segregation occurs and excess water is forced to the form faces. Such over-vibration may result in sand streaking and discoloration.

Splashing of concrete on forms

When concrete is splashed on the forms above the point of deposit and subsequently hardens, the wall in that area is likely to be rough and discolored. Splash boards are sometimes used but are not completely satisfactory as they do not prevent segregation of the falling concrete. Rectangular metal tremies of various lengths equipped with a hopper at the top through which the concrete is placed will avoid both splash and segregation. The tremies should be spaced not more than about 9 ft apart. Sometimes splashing is the result of mortar being thrown from vibrators as they are lifted out of the concrete and raised to the placing platform. This can be prevented by shutting off the power as the vibrator is withdrawn from the concrete.

FINAL OPERATIONS

Curing

On this type of work one of the surest and best ways of curing the concrete is by leaving the forms in place. Whenever it is practical to do so, they should be left in place for four or five days. By that time the concrete is hard enough so that it is not scuffed nor are corners so likely to be broken off during form removal. If forms are removed earlier, other means must be provided to keep the concrete wet or damp for at least five days if a durable concrete is to be expected.

Patching and finishing

When the forms are first stripped, the concrete does not always present a pleasing appearance as there may be pock marks, air holes, fins and some discoloration. Some of the latter will bleach out upon exposure

but a simple clean down with cement grout will do wonders for the appearance. The projecting fins and other blemishes extending beyond the face of the wall are removed with a carborundum stone or abrasive hone and then a grout of 1 part portland cement to 1½ parts of very fine sand (sand passing through a fly screen will do) is applied. From ⅓ to ½ of the cement should be white cement.

The grout is mixed to the consistency of a heavy paste. After the wall is wet down, the grout is applied uniformly with a stiff fiber brush, completely filling all air voids and holes. Immediately after the grout has been applied, the wall is floated with a wood or cork float. A very fine abrasive hone can also be used. The grout is then allowed to harden partially, the exact time depending on the weather. When it has hardened sufficiently so that it will not be pulled out of the holes, all grout is scraped from the surface of the wall with the edge of a steel trowel. After the wall has dried thoroughly, it is rubbed with a dry piece of burlap to completely remove all of the dried grout or film. There should be no visible film left on the surface. Grout should not be left on the wall overnight. All cleaning of any one panel of concrete should be completed the same day.

Patching mortar used to fill tieholes and imperfections should be of the same mixture of white and gray cements, but should be a stiffer mixture and the cement and sand proportions should be the same as those used in the concrete. Patches should never be steel troweled, but may be finished with wood or cork floats. Allowing the mixed patching mortar to stand for an hour or two before using it reduces the amount of shrinkage, but water must not be added in remixing it.

CONCLUSION

No matter how carefully and completely plans and specifications are prepared, craftsmanship on the job is essential to the perfection of any structure regardless of the construction material. Architectural concrete is no exception to the rule but neither is it more difficult to produce quality construction with concrete if proper attention is given to details and methods on the job.

In September **Letters from Readers** replaced **Job Problems and Practice,** with broader scope, more flexible form, and we hope more adaptability to the thoughtful, busy reader's use, as a forum for opinions, experiences and inquiries within the field of ACI activity. A letter does not need the studied treatment of a major paper to present valuable ideas for ready access to kindred minds.

Letters from Readers*

BY WAY OF SYNOPSIS

RAWSON F. STAGG inquires about the production and use of colored aggregates in concrete. The Editor provides a partial answer—other comment, particularly on production, is invited.

ROBERT J. WENING, JR., seeks data on the effect of oils on concrete.

Ceramic and Glass Aggregates for Colored Concrete (LR 45-15)

RAWSON F. STAGG, Sheffield, England, asks for information on colored aggregates in concrete

Can you tell me what results have been achieved in the field of ceramic and glass aggregates for colored concrete? Particular information is desired on their production and use.

The best known work with colored aggregates is that in architectural concrete by the late John J. Earley. The following are extracts from Mr. Earley's papers published by ACI.

The concrete was made of Potomac River sand selected to contain particles varying in color from white to yellow to brown and crushed and arranged in size so that at a distance of 100 ft the surface of the concrete would appear uniform in color and texture. The general hue

*A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 7, Mar. 1949, *Proceedings*, V. 45. Separate prints are available at 35 cents each. Address 7400 Second Boulevard, Detroit 2, Mich.

of the concrete is yellow red of bright value and weak chroma (YR 7/2) as noted by the Munsell System.[1]

These slabs were composed exclusively of carefully selected quartz, equally beautiful blue and yellow ceramics, black and red vitreous enamel and gold. These rare substances were crushed between great steel rolls and carefully screened to exact size. All dust was removed. No pigments were used, nothing but pieces of beautifully colored stone and stone-like materials, portland cement and water.[2]

The materials selected were as follows: The aggregate of the black base was a combination of black glass and two values of black ceramics and a vivid red glass. Black glass was used to give brilliance to the surface, but it could not be used alone because in full light too many facets would show specular lighting which makes them white and by that much brightens and grays the whole surface.

The materials for the first two courses were yellow quartz pebbles from the Potomac River and rose quartz pebbles from North Carolina, and two values of yellow ceramics. Experience with our first studies of these two lifts showed that the desired range of values for the whole tower would not be available in yellow quartz but that the intermediate values which we needed could be obtained in rose quartz....To the varying quantities of the above materials which governed the colors of the first two lifts, yellow ceramics were added to maintain the desired strength of chroma. In the next several courses the yellow ceramics were omitted and the yellow and rose quartz were gradually replaced by a less colorful quartz pebble found in Virginia, and by an opaque white quartz from South Carolina. For several more courses the less colorful quartz was replaced by a clear transparent quartz until all was white, and finally to the white material cobalt blue ceramics of high value and very weak chroma were added. The cements were normal white cement and gray cement from the same plant. They were blended or used separately as the needs required.

The aggregates were crushed and screened into two uniform sizes. The two sizes were recombined with portland cement in a proportion by weight of 1:1.11:3.2 or by volume of 1:1:3 and approximately five gallons of water. This mix was carefully cast in molds without disturbing its uniformity. When set, but before it had thoroughly hardened, the molds were removed and the surface of the concrete was brushed with steel wire brushes until the aggregate was evenly exposed over the whole surface. Subsequently the surface was cleaned with a weak solution of muriatic acid.[3]

[1] Earley, John J., "Building the Fountain of Time," ACI *Proc.* V. 19, 1923, p. 189.
[2] Earley, John J., "Mosaic Ceilings, U. S. Department of Justice Building," ACI JOURNAL, May-June 1935, *Proc.* V. 31, p. 557.
[3] Earley, John J., "Edison Memorial Tower", ACI JOURNAL, May-June 1938, *Proc.* V 34, p. 589.

In an informal conversation, Mr. Earley once said that he went to Italy and found glass makers from whom he obtained a considerable shipment of glass "pancakes" made in specified hues—a wide range of colors. In this country he crushed the pancakes for a graded aggregate for his "plastic mosaic". Similarly, in this country he obtained ceramics.

—Editor

What is the Effect of Oils on Concrete (LR 45-16)

ROBERT J. WENING, JR., Dayton, Ohio asks for information.

In a reinforced concrete building used for newspaper printing, a machine is setting in the center of a floor slab. This machine is excessively lubricated with a medium grade oil and over a period of approximately 7 years this excessive oil has been seeping through the floor slab, increasing a bit more all the time.

Information is desired regarding damage that can be done to concrete by various oils. We are hindered in removing a test core by the limited space around this machine.

Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

A new material for three-dimensional photoelasticity

M. M. Leven, *Proceedings*, Society of Experimental Stress Analysis, 1948, V. 6, No. 1, pp. 19-28.

Applied Mechanics Reviews Nov. 1948 (Frocht)

The progress of three-dimensional photoelasticity has in large measure been retarded by the lack of a suitable material for models. The best photoelastic material available until now was Bakelite BT-61-893. However this material suffers from several limitations which make it inadequate for many three-dimensional studies. Chief of these is the limitation of size. Since it is available only in plates approximately 12 in. long, 6 in. wide, and 1⅛ in. thick only the simplest three-dimensional models can be made. Another serious difficulty has been edge effects or time stresses. The techniques which proved successful in the elimination of these stresses in two-dimensional problems cannot be employed in three-dimensional problems. Further, at 260 F, the temperature necessary for frozen patterns, the Bakelite undergoes large deformation at even moderate stresses, so that the shape suffers serious distortion.

Fosterite, the new material developed by the Westinghouse Research Laboratories, is superior in each of the above respects. It already has been cast successfully in cylinders 30 in. long and 6 in. in diameter, and larger specimens can be expected. It is, for all practical purposes, entirely free of time stresses, and its deformations are relatively small. Stress patterns taken 30 days apart show only traces of time effects. The ratio of the modulus of elasticity to the tensile fringe value is twice as great for one type of Fosterite as for Bakelite. Another advantage of Fosterite is found in the short time necessary to cure it. The new material requires only one month compared to about eighteen months for the Bakelite.

Fosterite has excellent transparency, is easily machined, and at a temperature of about 100 C is perfectly elastic, with no measurable creep. It should, however, be noted that at room temperature it has a high rate of optical creep. For this reason the author does not recommend Fosterite for studies at room temperature, where Bakelite is still the best material.

In addition to describing the chemical, mechanical, and optical properties of Fosterite the author gives quantitative examples showing the reliability of this new material. The development of Fosterite represents a contribution of major significance to experimental stress analysis.

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Effect of delayed mixing of prebatched moist aggregate and cement on the strength and durability of concrete

W. H. Price and J. W. Robison, Preprint of a paper delivered at the A.S.T.M. 1948 Annual Meeting. Reviewed by George B. Wallace

When job conditions require that batched moist aggregate and cement be held for a considerable length of time before mixing, partial hydration of the cement in contact with the moist aggregate takes place. Unless sufficient quantities of cement are added to the batched materials when the mixer is charged to compensate for the hydrated cement, the resulting concrete will have less strength than concrete made from similar materials immediately mixed after batching. If the batched materials are delayed more than 6 hours before mixing, the resulting concrete wil also be less durable.

Four series of tests are described wherein cement was batched with moist aggregate and left covered with burlap for various intervals of time before mixing. In all test series the cement was completely intermingled with the moist aggregate when they were batched. The amount of free moisture in the aggregate was about 1.8 percent in the gravel and 7.5 percent in the sand. The slump of the concrete was held approximately at 3 in.

Some of the batches were mixed after delays up to 24 hours without adding cement at the mixer with average strength decrease in the resulting concrete of 4 percent per hour of delay. With delays up to 6 hours other batches were mixed with 5 percent additional cement for each hour of delay producing concrete of constant strength up to 3 hours delay with an increase of 260 psi per additional hour up to 6 hours where the tests were discontinued. In the final series a sufficient amount of cement was added at the mixer to maintain a constant ratio with the water necessary to produce a 3-in. slump. From the compressive strength of the concrete produced under these conditions the authors estimate that 100 percent additional cement would have to be added at the mixer to prevent strength loss with a delay of 24 hours.

Excessive bleeding occurred in the concrete with the longer periods of delay.

The results of freezing and thawing tests on all series indicate that no appreciable loss of durability occurs with delays up to 6 hours. Two series actually showed an increase in durability for the first 2 hours of delay. The maintenance of durability in spite of strength loss due to delay may be attributed to an increase in air content with delay. Measurements with a pressure-type air meter indicated from ½ percent to 1 percent increase in the air content for mixes delayed 4 hours or longer over the zero delay control mixes.

On the basis of the above tests the authors question the use of materials where the delay between batching and mixing exceeds 6 hours.

Studies on the properties of fresh concrete

Bulletin No. 12, Swedish Cement and Concrete Research Institute (Stockholm), 1948. 34 pp. Reviewed by Eivind Hognestad

Two papers report progress of a basic research project. No definite conclusions are advanced, and the contents are thus of principal interest to a research worker. An earlier report on this investigation is "Development of Fluidity and Mobility Meters for Concrete Consistency Tests," by A. G. Eriksson *Proc.* No. 12, Swedish Cement and Concrete Research Institute.

Properties of fresh concrete

E. Forslind

The workability of concrete, its definitions and measurements are discussed. The viscous properties of concentrated hydrophilic colloidal suspensions and gels are related to the properties of cement paste, mortar, and concrete. Some general principles

considered important for workability measurements are discussed. It is considered desirable to measure the viscous properties at as small deformations as possible.

Determination of the fluidity of concrete and cement
S. G. Bergstrom

An apparatus for the determination of the fluidity of fresh concrete is described. The upper end of a vertical axle is held rigidly by a bracket while the lower end is submerged in and coupled to the concrete by a special device. Torsional oscillations are initiated and measurements of the following damped oscillations are made by means of a photoelectric cell and an oscillograph. The inverse value of the logaritmic decrement is given the term "fluidity". Results obtained by measurements on concrete and cement pastes of varied compositions are reported and compared with results obtained by the use of common methods for the determination of consistency. The test values are discussed with regard to their significance and the sensitivity of the various apparatus.

V. Bährner and P. Hallström present short but pertinent remarks in a discussion.

Report of committee on frost heave and frost action in soil

Bulletin No. 14, Highway Research Board, 1948, pp. 1-11.

This report is a summary of the results of field installations made on projects in Minnesota, Indiana, Michigan and Massachusetts in which calcium chloride was used as a freeze-proofing agent. The results of these field experiments have not been particularly encouraging since in most instances there has been no diminishing of the original amount of heave. In contrast, however some reports indicate some benefit and in one instance a complete cure. Since laboratory experiments have indicated that a moderate concentration of calcium chloride in the soil will practically eliminate frost heave, it is the committee's opinion, which is based in part on the experimental data, that adverse water conditions have leached the chemical from the soil thus making the treatments ineffective in some instances. A program for additional study is being developed.

Jacked-in-place pipe drainage

Bulletin No. 14, Highway Research Board, 1948, pp. 21-40.

This paper summarizes the practical development of installation methods and evaluates empirical and theoretical resistances which may be used as a guide to the design of jacked-in-place drainage and other pipe. The procedure of jacking pipe for drainage connections and as sleeves to carry sewer, water, gas and electrical utilities under heavily travelled roadways and through high fills has been developed to a point where costs and construction requirements can be closely approximated.

Report of committee on soil cement roads

Bulletin No. 14, Highway Research Board, 1948, pp. 12-17.

This report reviews the maintenance costs and conditions in service of soil-cement roads, as inspected and reported to the committee by the several states. The construction details, age, traffic and present condition of the soil-cement base of the projects provide a complete history. Many variables in the construction, climate and traffic indicate that 6 in. of soil-cement base properly constructed is entirely adequate for the purpose. Other uses of soil-cement mixtures and cement-modified soil binders for soil aggregate mixtures were considered by the committee and further studies recommended.

Production of lightweight concrete aggregates from clays, shales, slates, and other materials
Report of Investigations 4401, U. S. Bureau of Mines. 121 pp.

Use of lightweight aggregates in constructing large concrete and steel buildings mean a saving of steel since the lighter concrete makes possible the use of less steel. Considered better suited to withstand the shock of earthquakes than standard concrete, lightweight concrete also can be used to build extra stories on buildings which cannot support the additional weight of standard concrete and essential reinforcing steel.

While production of lightweight aggregates—needed for all types of structural uses—is widespread, certain areas, notably east of the Mississippi, have only a few plants, and many plants are in need of suitable equipment. Besides determining usable raw materials, the Bureau studied the best type of equipment to use and developed satisfactory processes for producing good light-weight products.

At Tuscaloosa, Ala., the Bureau investigated clays; at Norris, Tenn.—shales; and at College Park, Md.—slates and other potential raw materials. Materials were classified as to their bloating or expanding characteristics—necessary to the production of aggregates. Not only were natural bloating clays and shales studied, but means were developed to cause most non-bloating materials to bloat controllably. Studies were made of the cause of bloating and definite bloating agents were added in various ways. The best products resembled a foam and not a sponge, and the strongest-foamed products were those with the smallest sealed-in bubbles of gas.

The appraisal of terrain conditions for highway engineering purposes
Bulletin No. 13, Highway Research Board, 1948. 92 pp.

The eight papers and appendix discuss the various phases of terrain analysis as related to highway location and construction.

The services of the U. S. Geological Survey available to the highway engineer are enumerated. Information on map coverage, data on horizontal and vertical control and indexes of aerial photographs and mosaics available from state and federal agencies are maintained by this agency.

The influence of land form on design, construction and maintenance of highways is discussed. Examples are given of the application of geological data to these problems. An air-photo pattern method is described which identified major outcrops in the Triassic area in Albemarle County, Virginia. The basic requirements for an engineering geological map are emphasized and their application to the preparation of a Montana map given.

Engineering soil groupings used in New York State are discussed along with the development of soil maps which constitute a basis for the planning, control and review of soil work in the design and construction of highways.

The application of geology and seismology to highway location and design in Massachusetts is presented. Complete and detailed geological studies embodied in colored geological maps are a valuable aid in design and construction of highways and bridges. Seismic soundings together with their geological interpretations are used to predict the depth of bedrock and the nature of the overburden. Information dealing with sub-soils and ground water aids in determining alignment, drainage facilities and bridge foundations.

Practical examples are given of highways located and designed by applying information obtained by geological interpretation and seismic exploration. An appendix describes in detail seismic tests and their interpretation on one project.

The present state of the theory of reinforced concrete (Vom Gegenwartstand der Stahlbetontheorie)

R. Saliger, *Oesterreichische Bauzeitschrift* (Vienna), V. 3, No. 11, Nov. 1948, pp. 170-177. Reviewed by Rudolph Fischl

A brief introduction deals with the limitations of the design in the elastic range and the uncertainty of the n-ratio. Next the author discusses the brittle and plastic rupture of reinforced concrete beams. The brittle rupture is the rule in under-reinforced beams where the resistance of concrete to flexure seems to be greater than the resistance to the bending of steel, hence this beam does not act any differently than concrete without reinforcement or other brittle materials such as stone, glass or wrought iron. These materials will break suddenly after the first crack has appeared.

The plastic theory received special attention in Europe and the United States during the past twelve years. Professor Saliger gives references to the extensive literature concerning that subject, including papers by Jensen, Whitney and others representing this country. A short development of the plastic theory is given, discussing heavily, medium and lightly reinforced sections and the application to design. Five graphs and one table are included. The author indicates that the plastic theory is ready to be included in the codes and is applicable to practical design. It eliminates the concept of allowable stresses and enables the design to be made with the required factor of safety, based on the ultimate strength of concrete and steel. It permits use of the full strength of material and introduction of a uniform safety factor, thus promoting a more economical design.

Use of special cements as construction material for dams in France. (Emploi des ciments speciaux pour barrages en France)

J. Cleret de Langavant, Center D'etudes et de Recherches de L'Industrie des Liants Hydrauliques, Paris, *Publication Technique* No. 6, July 1948. Reviewed by Ivan M. Viest

Provisions were made in France before the war to use slag cement as a construction material for several important dams. Because of difficulties brought on by the war, it was necessary to modify this program and excellent results were obtained by using artificial cement containing very small quantity of aluminum and made according to the "double baking process". Available quantities of this cement are, however, too small for use as common construction material for all the dams planned.

The attention of the French Design Service has been drawn to the utilization of a new process for manufacturing the cement on the job by wet grinding of the slag. With this process, while consuming the same amount of power as for grinding normal slag cement, it is possible to produce a cement of specific fineness about double that normally obtained. This cement has the proper qualities of a normal slag cement, namely: durability and low hydration heat. Moreover, unlike ordinary slag cement, it has great workability owing to its high degree of fineness and offers the possibility of making strong and watertight concrete.

The author discusses different chemical and mechanical properties of both the "double baking process"and wet-ground slag cement and of concrete made there from. He also brings out the economy of use of slag cement wet ground on the job.

Symposium on usefulness and limitation of samples

Proceedings, A.S.T.M., V. 48, 1948. 40 pp.

The symposium consists of three papers on sampling together with discussions given at the 1948 annual meeting of A.S.T.M. The laws relating to sampling, the amount of testing necessary to assure general acceptance of good material and the rejection of poor material, and the reduction of inspection when quality is consistently good are considered.

Summary of heat insulation properties of hollow concrete blocks and perforated bricks

HJALMAR GRANHOLM, *Transactions*, Chalmers Univ. of Tech., Gotherburg, Sweden, No. 80, 1948. AUTHOR'S SUMMARY

The usual methods of determining the thermal insulation of hollow blocks often gives erroneous and decidedly misleading results. In the present paper a simple and reliable electrical analogue is described, which allows a study of the heat resistance of hollow blocks of various types and manufactured of material with arbitrary thermal conductivity.

Among the more than 100 different types of hollow concrete blocks on the Swedish market a number of characteristic representatives are analysed. To compare the various makes of hollow blocks a quality number is introduced, indicating how many times the thermal insulation is increased owing to the hollows and air spaces. The quality number lies between 1.10 and 4.50 for the current types of blocks depending on the shape and number of the spaces. It is technically possible to reach quality numbers from 5.0 to 6.0. The results obtained are compared with tests on full scale walls.

The current types of perforated bricks with 75, 105 and 119 square perforations are also studied. The paper contains a suggestion to improve these types of bricks by making the perforations in the shape of narrow rectangles instead of the now usual holes ¾ x ¾-in. The quality number of the brick can be increased in this way from 1.45-2.00 up to 2.00-3.50. according to the type of brick used. A comparison is made with tests on full scale brick walls and the tests are analyzed in detail.

A survey of pavement pumping in Illinois

Research Reports No. 10—1948 *Suppl.*, Highway Research Board. 48 pp.

Approximately 300 miles of the principal highways in northeastern Illinois were the subject of an intensive pavement pumping study to determine the cause and extent of pumping and possible means of preventing its occurrence on future work.

Analysis of data on 78 projects confirmed the fact that free water must be present, axle loads must be heavy and frequent, and the subgrade soil must be capable of going into suspension to produce pavement pumping.

Design features such as pavement width and thickness, pavement reinforcement, expansion and contraction joints, joint spacing and fillers, and load transfer devices were found to have, in most cases, some influence on pumping. However, no single feature or group of features were found capable of preventing pumping under all conditions of moisture, traffic and soil.

The texture of the subgrade determined its susceptibility to pumping. Increased compaction generally reduced pumping. Soil containing more than 55 percent sand and gravel (material retained on the No. 270 sieve) was found to prevent pumping. Furthermore, granular subbases will prevent pumping.

Prompt and continued pavement and road maintenance was also found to be effective in controlling and reducing pumping. Repeated subsealing effectively controlled pumping.

Distribution of load on a group of piles

A. L. L. BAKER, *Concrete and Constructional Engineering*, V. 43, No. 11, Nov. 1948, pp. 335-342. Reviewed by GLENN MURPHY

The author presents an analysis for load distribution based on the assumption that the pile cap is rigid, that the piles provide resistance to bending, that all piles have the same area and the same vertical component of length, that no settlement occurs, that a plane of symmetry exists for the group and that the loads are applied on the plane of symmetry. Arrangements and other practical details are discussed briefly.

Recommended practice for snow removal and treatment of icy pavements

Current Road Problems No. 9-2R (Second Revision), Highway Research Board. 17 pp.

Since the publication in December 1947 of the first revision of the original Wartime Road Problems Bulletin No. 9, Recommended Practice for Treatment of Icy Pavements, a Highway Research Board committee has studied prevailing practices in removal of snow. This revision presents recommendations for organizational personnel, training program, drift control, markers for drainage and structures, drainage facilities, plan of operation, use of weather reports, night patrols, public warnings, types of equipment, lighting and color scheme of equipment, dispatching of equipment, control, plowing, and for reports on snow removal.

The section on ice removal is modified slightly from that given in the December 1947 edition.

Survey of papers on elasticity published in Holland 1940-1946

C. B. Biezeno, *Advances in Applied Mechanics*, Academic Press Inc., New York, 1948, V. 1, pp. 105-170.

Applied Mechanics Reviews Nov. 1948 (Sternberg)

This instructive survey should be of great interest to research workers in the fields of theory of elasticity, elastic stability, plates and shells, general theory of structures, airplane structures, and experimental stress analysis. A major portion of the article is devoted to publications of the laboratory for applied mechanics of the Technische Hoogeschool at Delft. The papers discussed in this first section are based on investigations conducted by the author, his collaborator J. J. Koch, as well as other associates and students of the author. The second section deals with publications of the department of vibrations and photoelasticity of the Dutch Central Institute of Technical Scientific Research. This section is followed by a summary of publications of the National Aeronautical Research Laboratory at Amsterdam. The author concludes his survey by references to the treatises Technische Dynamik, by C. B. Biezeno and R. Grammel, Springer, Berlin, 1939, and to Plasticiteitsleer (theory of plasticity), by F. K. Th. van Iterson, A. E. E. Kluwer, Deventer, Holland, 1945. A complete bibliography of all papers and books mentioned is given at the end of the article.

Buckling of trusses and rigid frames

G. Winter, P. T. Hsu, B. Koo, and M. H. Loh, *Bulletin* No. 36, Cornell Univ. Engineering Experiment Station, April 1948, pp. 1-63

Applied Mechanics Reviews Nov. 1948 (Allen)

This paper begins with brief summaries and reproductions of standard methods of stress analysis for rigid-jointed frameworks, developing the general slope deflection equations, and describing the methods of moment distribution and of end restraints.

It then extends these methods to allow for the effects of axial load in each member and for the restraining action, at its ends, of similar members attached to it there. For this purpose the authors develop formulas for an "effective flexural rigidity" considering the action of the axial loads. The paper is primarily concerned with the approximate trial determination of the critical magnitude of a given distribution of load, that is, the magnitude which would cause buckling in the structure.

Prismatic thin-slab structures of one span—II

A. J. Ashdown, *Concrete and Constructional Engineering*, V. 43, No. 11, Nov. 1948, pp. 347-352.

Reviewed by Glenn Murphy

This installment contains two examples: one, for a slab with both edge ties supported; the other, with only one edge supported. Stresses around openings for roof lights are discussed.

Plant for a cementlike product, Hydromentwerk Rummelsburg, Berlin

E. Strauch and H. Starck, *Die Technik*, V. 3, No. 3, 1948, pp. 117-122. Ceramic Abstracts

Cement will remain scarce for a long period in Germany, and endeavors to utilize waste in industry with the present scarcity of coal necessitates the use of any material available. The fly ash from coal at the Klingelnberg power station is discussed. A new cementlike product called Hydroment consists only of lime and ash. It's composition is not exactly that of average portland cement, as the silica content is too low and the alumina and iron oxide contents are too high. In spite of the differences in chemical composition, its strength properties may approximate those of portland cement. Investigations on this material started with the end of the last war, and at present a plant is being erected.

A 1,500,000 gallon elevated water tank at Stoveley

Concrete and Constructional Engineering, V. 43, No. 11, Nov. 1948, pp. 343-346. Reviewed by Glenn Murphy

Construction details of what is thought to be the largest elevated cylindrical tank in the world are described. The tank has an average internal diameter of 130 ft and a depth of water of 18¼ ft, with a minimum head of 21 ft above the foundations. To reduce secondary stresses the outer 20 ft of the 12-in. flat-slab floor is thickened to form a ring beam to which the wall is attached. Columns are more closely spaced under the ring beam than elsewhere, to reduce torsion in the wall. The design of the flat slab conforms to the Code for Reinforced Concrete of the Building Industries National Council. Working stresses of 900 psi in the concrete and from 12 to 16 ksi in the steel were used. Construction details are described.

The determination of the proportions of concrete

L. J. Murdock, *Cement and Lime Manufacturing* V. 21, No. 5, 1948, pp. 91-96. Building Science Abstracts

Current methods for determining the cement content in a sample of mixed concrete are lengthy and laborious, since they involve estimation of lime or silica, separation of coarse and fine materials by wet sieving, and evaporation of excess water from the fines. The method here described eliminates the evaporation of large amounts of water; it is based on the determination, by a hydrometer, of the specific gravity of a suspension in water of the material passing a 100-mesh sieve. The effect of dust passing the 100-mesh sieve, as well as cement particles, is allowed for. The method is accurate to within 4 percent.

Manufacture of prestressed concrete bridge girders

Concrete and Constructional Engineering, V. 43, No. 11, Nov. 1948, 1948, pp. 353-355. Reviewed by Glenn Murphy

The construction of 64 prestressed beams 27 to 29 ft in length is described. All of the beams are 32 in. deep and are I shaped, reinforced with 42 hard-drawn steel wires 0.20 in. in diameter. The beams were cast in two groups of 32 each placed end to end so that the reinforcing wire was stretched in 930 ft lengths. Jacks were used to develop the required stress of 70 tons per sq in., which required an elongation of 60 in. The prestressing force was maintained for 10 days, the load was released at the jacks and the reinforcing wires between beams cut with a torch.

ACI News Letter

Vol. 20 No. 7 JOURNAL of the AMERICAN CONCRETE INSTITUTE March 1949

Views showing damaged piers and dam after Prepakt restoration

PREPAKT REPORTS ON...

OPERATION FROSTBITE

The repair work done on the Latchford, Ontario, dam was appropriately dubbed "Operation Frostbite" due to the extensive damage to the existing dam piers caused by freezing and thawing action. Disintegration of the concrete in the underwater and exposed sections of the piers, on both the up and downstream side, was so extensive the safety of the dam as well as proper manipulation of the stop log timbers were affected.

With the repair work being done in severely cold weather, PREPAKT methods and materials were successfully used to restore the piers to a new and fully useful condition.

The extremely high bonding strength of Prepakt Concrete plus its high resistance to frost damage and other weathering factors, assures a long and maintenance-free life for this dam structure.

In addition, PREPAKT methods and materials provided a considerable production economy in this exacting job.

THE CONCRETE WITH PREPAKT EXTREME DURABILITY

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ACI

News Letter Contents

March 1949

On the Cover — home office of General Petroleum Corp., Los Angeles; largest office building in Southern California. Three of four walls are of hollow construction, lightweight pumice concrete placed on steel mesh by the Gunite process permitting a reduction of 8300 tons in dead load.

The New York Convention 3

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Just a flash about ACI's New York Convention

Feb. 23-25, 1949

Most of this JOURNAL was pretty much "all sewed up" at the printers before the 1949 convention got under way, and it remains for the April issue to record an adequate review of the annual event and some of the papers in full. Meanwhile these notes:

With two morning, three afternoon and no evening sessions—except as several committees put in considerable overtime (beyond the half-day period scheduled for committees)—those in attendance apparently had a good deal to say that went unsaid for want of time. Discussion on occasion was lively. This and other considerations have given rise to the possibilities of concurrent sessions at future conventions—problems of structural design perhaps going along in one auditorium and matters of on-the-job-construction in another. How would ACI Members like that?

Convention registration got away—with makeshifts—under the handicap of waiting while two staff scouts tore through strike-bound express warehouses hunting and finding part of the forms and paraphernalia shipped from Detroit. Still, it was a good natured line of registrants that waited.

Continued on p. 4

Frank H. Jackson led the first general session Wednesday afternoon which developed a good deal of interest and discussion on pavement durability and air entrainment.

The structural session Thursday morning in brief—perhaps too brief—presentations of papers aimed chiefly at the design of slabs and footings; and posed problems for future labors of ACI's Committee 318, Standard Building Code.

More than 300 sat down to the luncheon Thursday noon at which President Blanks bestowed the honors of annual awards. At that hour a group mostly of ladies were off for a United Nations tour.

An invitation to ACI members to attend the "First International Civil Engineering Congress" in Mexico City, April 30 to May 7 was ably presented by Mr. Francisco Gomez-Perez.

A moment of silence in memory of Arthur J. Boase on Thursday afternoon, preceded the tellers report of new officers: President Herbert J. Gilkey; junior Vice President Harry F. Thomson; new Directors Harmer E. Davis, H. C. Delzell, Nathan M. Newmark, R. W. Spencer. Retiring President Blanks looked into a crystal ball and called forth the concrete of 50 years hence in some imaginative predictions. The "ball" was really an "8 ball" which he set in front of his successor, Professor Gilkey. Next, the users and producers of ready mixed concrete had a lively inning following Stanton Walker's paper on "control." Discussions, written and extemporaneous, were numerous and will make interesting reading later. Committee 616's report on the Application of Portland Cement Paint to Concrete Surfaces was adopted as an ACI Standard, subject to ratification by letter ballot.

Friday morning the Research program —chairman, S. J. Chamberlin—made a well packed period of short contributions. (The executive group of that committee has proposals for the Technical Activities Committee for future greater usefulness.)

Friday afternoon two papers (both in this issue) took the problem of architectural concrete from the drawing boards to the job; lightweight aggregate concrete had consideration; a resumé of the growth of the concrete masonry industry was presented.

Thanks were extended to the New York Convention Committee, M. J. McMillan, chairman, for much valuable preliminary labor to make the convention the success it seemed to be.

528 persons registered—and others got in without registering.

"A lot of good solid meat," said some of the those present of the convention's program fare.

Next year it's Chicago, week of February 20, 1950.

It is important, too, to note that the Technical Activities Committee adopted important preliminary measures to give more breadth and coverage to future JOURNALS—measures of progress toward the "goals" adopted by the Board of Direction last October (see ACI JOURNAL Nov. 1948, p. 3).

For the
COMPLETE CONVENTION STORY
see
APRIL NEWS LETTER

Who's Who This Month

B. D. Tallamy

B. D. Tallamy describes New York State tests to overcome deterioration of pavements in his "Control of Concrete Pavement Scaling Caused by Chloride Salts," p. 513. His paper was presented at the 45th annual convention February 23, 1949. Mr. Tallamy is superintendent of public works of the State of New York. With that department since 1945 he has been successively deputy superintendent and chief engineer with general supervision of the state's $840,000,000 postwar public works program. Previously he was chief engineer of the Niagara Frontier Planning Board and a member of the engineering firm of Fretts, Tallamy, Senior and Forrestel. Early experience was as field engineer and party chief for waterworks, sewage plants, roads and bridges in western New York which led to specialization in sanitary and general municipal engineering.

Mr. Tallamy is a graduate of Rensselear Polytechnic Institute with the degree of Civil Engineer.

Charles M. Noble

State highway engineer for New Jersey, Charles M. Noble, reports "Experience with Air-Entraining Concrete in New Jersey," p. 521. His paper was presented at the 45th annual convention February 23, 1949.

During World War II Commander Noble served with the Civil Engineering Corps, U.S.N.R. as commanding officer of construction units engaged in base construction, air fields, docks, and general construction in Alaska and the Aleutians. In addition, he reconnoitered Japanese-occupied China for engineering information, served on Admiral Nimitz's staff, and was deputy director, Advance Base Department, Bureau of Yards and Docks, Washington, D. C. For his outstanding service he was awarded the Legion of Merit and the Bronze Star medals.

Before entering the service Mr. Noble was highway engineer for the Pentagon Building and the Pennsylvania Turnpike and for 13 years was engineer for the Port of New York Authority on bridge and tunnel projects including Outerbridge Crossing, Goethals, Bayonne, and George Washington bridges; and the Lincoln Tunnel. In addition he was connected with engineering work in Florida, Alabama, North Carolina, and Kentucky.

His paper "The Modern Express Highway" was awarded the A.S.C.E. Arthur Wellington Prize and the Boston Society of Engineers gave their Clemens Herschel Prize for his paper "The Factor of Safety in Highway Design."

Mr. Noble is a director of the American Road Builders Assn. and a member of A.S.C.E., New Jersey and National Society of Professional Engineers, and the Highway Research Board.

John J. Hogan

"Design Details for Architectural Concrete," p. 529, was presented by Mr. Hogan at the 45th annual convention, February 25, 1949. With 25 years experience in the civil engineering field he has been for the past six years regional structural engineer in the New York office of the Portland Cement Association. Previous associations were with Defense Plant Corp. as assistant chief, materials section; Parsons, Klapp, Brinckerhoff and Douglas as structural engineer; New York Central Railroad Co. as designer and detailer; and other work in surveying and concrete production and placement.

Mr. Hogan received his Master of C. E. degree at New York University, is

a licensed professional engineer in New York State and a member of A.S.C.E., the Bronx Chapter of Professional Engineers, the Society of American Military Engineers, Tau Beta Pi and ACI.

E. B. Oberly

"Construction Practices for Architectural Concrete," p. 541, gives E. B. Oberly's recommendations for this type of work as a result of his experiences since 1924, as presented at the 45th annual convention February 25, 1949. Since 1940, with the exception of two years during the war, he has been with PCA where he is regional construction superintendent in the New York office. From 1936 to 1940 he operated his own contracting firm. He was a construction supervisor for a number of contractors in Arizona and California between 1924 and 1936.

Arthur Boase

ACI Director Arthur J. Boase, 56, for the past 16 years manager of the structural and railways bureau of the Portland Cement Assn., died of a heart attack in his office on February 9, 1949.

Mr. Boase joined PCA in 1930 as regional structural engineer in the Philadelphia district. A civil engineering graduate of the University of Colorado, Mr. Boase later received his master's degree from the University of Pennsylvania and spent his entire professional life in structural design work.

After completing his college work, Mr. Boase designed dams and tunnels in the Rocky Mountain region for eight years, principally for the Boston Colorado Power Co. Then for three years, as manager of the Fair Engineering Co. in Denver, he worked on the design and construction of reinforced concrete bridges in Colorado.

After serving on the staff of the civil engineering department of the Univ. of Pennsylvania, he was appointed professor of civil engineering to head a similar department at Pennsylvania Military College. He left this work to join PCA.

Mr. Boase was an internationally known authority in the field of reinforced concrete design and had long been active in ACI committee work. He traveled extensively for PCA in Europe in 1936, studying European practice in reinforced concrete design. In 1944, he was granted a leave of absence to visit South America and to write an extensive series of articles for *Engineering News-Record* on the design and construction methods of South American engineers and architects in the field of reinforced concrete building. He was selected as the man best qualified in his field to handle this assignment.

An ACI member since 1933, Mr. Boase was the author of numerous papers and had served on many committees. He had been a member of the Board of Direction since 1947. At the time of his death he was chairman of ACI Committee 318, Standard Building Code. He was also chairman of the executive committee of the A.S.C.E. Structural Division and the A.S.C.E. committee on Masonry and Reinforced Concrete. He was a member of the Reinforced Concrete Research Council and the joint committee on Standard Specifications for Cement and Reinforced Concrete. He was probably most widely known as chairman of Committee 318 and as author-chairman of the Reinforced Concrete Design Handbook prepared by ACI Committee 317.

Colonel McWilliams reappointed to Delaware position

Colonel William A. McWilliams has been renamed chief engineer of the Delaware State Highway Department. A veteran of both world wars, he served as assistant chief engineer before assuming his present duties in 1947 when W. W. Mack retired as chief engineer.

New Members

With 126 new member applicants approved for the month of January, less 33 losses (death, resignation and "dropped for non payment of dues") the January 31 membership total became 4,488 (a net gain of 93), and at this later writing ACI is well past the 4500 mark.

Among the 126 members listed are 79 individuals, 9 corporations, 23 Juniors and 15 Students.

Geographically, New York State leads the list of January newcomers with 14; California in second place with 11.

The next intermediate goal is 5,000 members. If our now more than 4,500 members should all miraculously become member conscious merely by passing to non-member friends and acquaintances some word of what ACI is doing for present members, we'd have to raise our goal sights in short order.

This is not a "drive" for members—drives fizzle out; it is merely healthy growth as word "gets around." Will you help see that word DOES get around.

Alabama

McCormick, Francis Miller, (Jr.) Box 124, Childersburg, Ala.

California

Balch, William Thomas, (St.) 2609 Ivy Drive, Oakland 6, Calif.
Ehlert, F. J., (Indiv.) 6562 Sylmar Ave., Van Nuys, Calif.
Flam, Bernard, (Jr.) 15025 Oxnard St., Van Nuys, Calif.
Fowler, Lloyd C., (St.) 2276 Shattuck Ave., Berkeley 4, Calif.
Horowitz, Saul Jr., (Jr.) Engineer Section, Hdqr. IX Corps, APO 309, c/o P.M., San Francisco, Calif.
Khoury, Esa Michael, (Indiv.) 2219 S. Raymond Ave., Los Angeles, Calif.
Lee, Robert G., (Indiv.) 1319 Powell St., San Francisco 11, Calif.
Sumpter, Leon W., (Indiv.) 1340 E. 6th St., Los Angeles 21, Calif.
Udell, Norris, (Indiv.) Road Commissioner, County of Mariposa, P. O. Box 74, Mariposa, Calif.
Vernova, John J., (Indiv.) c/o Bureau of Reclamation, P. O. Box 570, Tracy, Calif.
Voelker, J. F., (Indiv.) Riverside Cement Co., Box 832, Riverside, Calif.

Colorado

Anderson, Robert R., (Jr.) Box 216, Haxtum, Colo.
Chadha, Om Parkash, (Indiv.) c/o International Engr. Co., P. O. Box 356, Denver, Colo.
Chandler, Robert J., (Indiv.) 1920 Quince St., Denver 7, Colo.
Chopra, M. R., (Indiv.) c/o International Engr. Co., P. O. Box 356, Denver, Colo.
Kumra, Prem Nath, (Indiv.) c/o International Engr. Co., P. O. Box 356, Denver, Colo.
McPherson, Thomas Buchanan III, (Indiv.) Civil Engineering Dept., Colorado School of Mines, Golden, Colo.
Singh, Tirlochan, (Indiv.) c/o International Engr. Co., P. O. Box 356, Denver, Colo.

District of Columbia

Grimm, Clayford T., (St.) 4493 Quarles St., N. E., Washington 19, D. C.
Gupta, O. P., (Indiv.) c/o Embassy of India, 2107 Massachusetts Ave., Washington, D. C.

Florida

Coogan, Thomas P., (Indiv.) 1432 duPont Bldg., Miami 32, Fla.
Lewis, Robert H. Jr., (St.) Beta Theta Pi House, Gainesville, Fla.
Sayles, Francis Harvey Jean, (Jr.) 371 Dellwood Ave., Jacksonville, Fla.

Georgia

Martin, William Reid Jr., (Jr.) Engineering Dept., Georgia Power Co., P. O. Box 1719, Atlanta 1, Ga.

Illinois

Layman, Roy E., (Indiv.) 4538 West End Ave., Chicago, Ill.
Long, Thomas E., (Indiv.) Portland Cement Assn., 33 W. Grand Ave., Chicago 10, Ill.
Tanenbaum, Louis, (Indiv.) 1553 W. Madison St., Chicago 7, Ill.
Wilson & Co., Inc., (Corp.) 4100 S. Ashland Ave., Chicago 9, Ill. (M. J. Hess)
Zakharoff, Alexis A., (Indiv.) 6046 Dorchester, Chicago 37, Ill.

Indiana

Aurenz, Melville D., (St.) 12 N. 20th St., Lafayette, Ind.

Continued on p. 8

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Miser, R. Frank, (Indiv.) 1326 Maple Ave., Fort Wayne, Ind.

Iowa

Welden, Neil, (Indiv.) 516 Grand Ave., Ames, Ia.

Kansas

Antrim, Howard C., (Indiv.) 1313 Quindaro Blvd., Kansas City 2, Kans.
Ericson, Neil Adair, (St.) 113 W. Beloit, Salina, Kans.
Mason, John Leland, (St.) 957 Ridgewood, Wichita, Kans.
Rice, Warren Glenn, (St.) Greensburg, Kans.

Kentucky

Craig, Charles S., (Indiv.) 21 Glenway, Fort Thomas, Ky.

Louisiana

Everard, Noel James, (Jr.) 2614 Dreaux Ave., New Orleans, La.
Freeman, E. M., (Indiv.) Texas Eastern Bldg., Shreveport 23, La.

Massachusetts

Penzien, Joseph, (St.) Grad. House, MIT, Cambridge 39, Mass.
Rumberger, William D., (Jr.) c/o General Delivery, Scituate, Mass.
Seppala, Albert M., (Indiv.) 92 Granite St., Rockport, Mass.
Snyder, I. Sterling, (Jr.) 42 Eliot St., S. Natick, Mass.

Michigan

Crocker, John Gilbert, (St.) 1406 Erving Court, Willow Run Village, Mich.
Finley, Ewell W., (St.) 1373 Swansea Court, Willow Run, Mich.
Muskegon, City of (Corp.) Engineering Dept., City Hall, Muskegon, Mich. (Edwin Johnson)

Missouri

Crabb, William A., (Indiv.) c/o Black & Veach, Engrs., 4706 Broadway, Kansas City, Mo.

Nebraska

Blackman, James S., (Indiv.) 303 Bancroft, University of Nebraska, Lincoln, Nebr.

New Hampshire

Surowiec, Edward J., (Indiv.) 39 Highland Ave., Franklin, N. H.

New Jersey

Gottschalk, Werner, (Indiv.) 58 Chestnut Drive, Packanack Lake, N. J.
Sturm, Werner C., (Indiv.) 118 Walnut St., Roselle Park, N. J.

New York

Barron, Maurice, (Indiv.) 1 Old Mamaroneck Rd., White Plains, N. Y.
Copeland, Warren Ellsworth, (Indiv.) 127 Ossining Rd., Pleasantville, N. Y.
Fraioli, Joseph, (Indiv.) 65 Batavia Place, Harrison, N. Y.
Frey, Erwin U., (Indiv.) 2962 Decatur Ave., New York 58, N. Y.
Garrelts, J. M., (Indiv.) Civil Engineering Dept., Columbia University, New York 27, N. Y.
Gut, Tadius Julius, (Indiv.) 55-12 32nd Ave., Woodside, L. I., N. Y.
Hopper Products Inc., (Corp.) Portite Division, 12 E. 41st St., New York 17, N. Y. (Edward T. Campbell)
Hunter, Kenneth, (St.) 223 Scottwood Ave., Elmira Heights, N. Y.
Karpowich, Daniel B., (Jr.) 138 Rosa Rd., Schenectady, N. Y.

Kline, Thornton C., (Indiv.) 11 Malcolm Ct., Oceanside, N. Y.
Lacey, James F., (Indiv.) Apt. No. 1, 201 W. 108th St., New York, N. Y.
Nersesian, Seron S., (Jr.) 500 W. 175 St. New York 33, N. Y.
Perrone, Alfred J., (Indiv.) 1071 75 St., Brooklyn 28, N. Y.
Wolfson, William, (Jr.) 873 E. 26th St., Brooklyn 10, N. Y.

North Carolina

Yike, Joseph S., (Indiv.) 2010 Crescent Ave., Charlotte 7, N. C.

Ohio

Andrews, Vernon S., (Indiv.) 1430 Sulzer Rd., Euclid, Ohio
Bare, Ben K., (Indiv.) 584 E. Broad St., Columbus 15, Ohio
Carmody, Michael M., (Indiv.) 3350 Felicity Dr., Cincinnati 11, Ohio
Hersey, A. T., (Indiv.) P. O. Box 31, Oxford, Ohio

Oregon

Batterman, Erwin Edward, (Indiv.) 147 N. Commercial St., Salem, Ore.
Kenney, Robert Jackson, (Indiv.) 32 N. Central Ave., Medford, Ore.
McCormack Concrete Pipe Co., Hillsboro, Ore. (Mr. McCormack)
Willamette Hi-Grade Concrete Co., (Corp.) N. Portland Ave., Portland 3, Ore. (A. A. Riedel)

Pennsylvania

Blaw-Knox Construction Co., Chemical Plants Division, (Corp.) 321 Penn Ave., Pittsburgh 22, Pa.
Greenleaf, Lee M., (Indiv.) Aluminum Company of America, 801 Gulf Bldg., Pittsburg, Pa.
Ingram, William Robert, (Jr.) Alpha Portland Cement Co., 15 S. Third St., Easton, Pa.
Pien, Chung Ling, (Indiv.) 1111 Tennessee Ave., Pittsburgh 16, Pa.
Rennis, Frank George, (Indiv.) 7323 W. Chester Pike, Upper Darby, Pa.
Vaughn, Edward L. Jr., (Jr.) 5043 Cottage St., Philadelphia 24, Pa.

Tennessee

Howe, Warner, (Indiv.) 344 Josephine St., Memphis, Tenn.

Texas

O'Rourke, George P., (Indiv.) 1001 W. Commerce St., P. O. Box 5384, Dallas 2, Texas
Nutting, William Franklin, (Jr.) 2212 Lovedale, Dallas 9, Texas
Smith, E. C., (Indiv.) Box 587, Brownsville, Texas
VanDerHeck, W. W., (Indiv.) 244 Ross Ave., San Antonio 11, Texas

Utah

Mackie, Thomas T., (Indiv.) 1605 Maple Ave., Salt Lake City, Utah

Virginia

Hartless, Thomas Bernard, (Indiv.) Box 1J, Richmond, Va.
Kuhn, Coleman, (St.) "Westover" RFD No. 3, Charlottesville, Va.

Washington

Kitchin, Charles Edward, (Indiv.) c/o John Graham & Co., 1501 Dexter Horton Bldg., Seattle, Wash.
Waterhouse, Robert W., (Indiv.) c/o District Engineer, APO 942, P. M., Seattle, Wash.

West Virginia

Roit, Fiore Da, (Indiv.) Don Bosco Agricultural School, Huttonsville, W. Va.

Wisconsin

Spray-O-Bond Company, (Corp.) 2225 N. Humboldt Ave., Milwaukee 12, Wis. (R. F. Forrer)

Wyoming

Palmer, Fred Irving, (Jr.) P. O. Box 444, Laramie, Wyo.

Hawaii

Kunimoto, Yoshio, (Indiv.) 2021 Waiola St., Honolulu 27, T. H.
Morrison, Vaughn O., (Indiv.) Paia, Maui, T. H.

Puerto Rico

Puerto Rico Transportation Authority, (Corp.) P. O. Box 3508, San Juan, Puerto Rico (Salvador V. Caro)

Canada

Belanger, A., (Indiv.) 10678 Pelletier St., Montreal, North Que., Canada
Clark, Gordon, (Indiv.) 8108 114 Ave., Edmonton, Alberta, Canada
Dusel, Frank Joseph, (Indiv.) 2065 Reynolds St., Regina, Sask., Canada
Ellis, D. S., (Indiv.) Faculty of Applied Science, Queen's University, Kingston, Ont., Canada
Finlayson, Donald William, (Indiv.) 37 Roxborough St. W., Toronto, Ont., Canada
Jennings, D., (Indiv.) No. 1 Rd. Maint. Coy RCE, Whitehorse, Yukon Territory, Canada
McIntosh, Duncan Neil, (Indiv.) 513 Pigott Bldg., Hamilton, Ont., Canada

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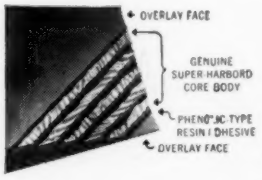

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Continued from p. 9

Colombia

Velasquez, Leonel Calle, (Jr.) Poblado, Calle 10, No. 33-06, Medellin, Colombia, S. A.

Costa Rica

Leon-Paez, Jose Maria, (Jr.) Apartado "Q," San Jose, Costa Rica, C. A.

Cuba

Garayta, Jorge A., (Indiv.) Milanes 208 C, Matanzas, Cuba

Denmark

Larsen, R. A., (Indiv.) Frydendalsvej 27, Copenhagen V, Denmark

England

Drinkwater, John, (Jr.) 245, Adswood Rd., Adswood, Stockport, Cheshire, England

United Kingdom Construction & Engineering Co., Ltd., (Corp.) Bridgewater House, 60 Whitworth St., Manchester 1, England (E. Copeland Snelgrove)

Wark, Donald Holme, (Jr.) 11, Mond Rd., Hr. Irlam, Nr. Manchester, England

Wegrowski, Jan., (Indiv.) 8, Lisgar Terrace, London W. 14, England

Guatemala

Saenz, Jose Eugenio Toledo, (St.) 2ª Ave. Sur No. 43, Guatemala City, Guatemala, C. A.

India

Banerjee, Robindra Nath, (Jr.) No. 16 Colvin Court, Howrah, West Bengal, India

Goswami, Mihir Kumar, (Jr.) P. 16 Old Ballygunge Rd., Calcutta 19, India

Ismaili, S. A., (Indiv.) S. S. E. Class I., c/o Sind P. W. D., Karachi, Pakistan

Mukerji, Syamsundar, (Jr.) P. O. & Vill. Ghola, 24 Parganas, West Bengal, India

Israel

Muller, L. S., (Indiv.) c/o B. Chaikin & E. Nesher, Architects, 1. Ben Jehuda St., P. O. Box 627, Jerusalem Israel

Malaya

Taylor, Gordon S., (Indiv.) 3, Avon Rd., Singapore, Malaya

Mexico

Cordero, Vicente, (Indiv.) Rio Amur 13, Col. Cuahtemoc, Mexico D. F., Mexico

Hiriart, Fernando, (Indiv.) Heriberto Frias No. 623, Mexico D. F., Mexico

Netherland West Indies

Bond, Oliver James, (Indiv.) Box 888, Lago Oil & Transport Co., Ltd., Aruba, N. W. I.

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WHAT WILL CALCIUM CHLORIDE DO FOR GOOD CONCRETE?

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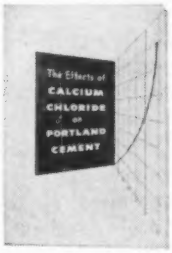

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Positions and Projects — ACI Members

Blanks speaks at Iowa State

Robert F. Blanks, ACI president, delivered two lectures January 17, 1949 at his Alma Mater, Iowa State College, Ames, Iowa. He addressed the Engineering Assembly on "Problems in the Design and Maintenance of Large Dams," summarizing U. S. Bureau of Reclamation experiences in irrigation, flood control and power development.

"The Technical Tools of Modern Engineering", was the subject of an illustrated lecture to the Iowa State College student chapter of A.S.C.E. ACI members in the vicinity were invited to meet and hear President Blanks.

Jay E. Jellick

Jay E. Jellick has been appointed manager of the Portland Cement Information Bureau, San Francisco, Calif., to succeed the late Ralph Rader, this position having been formerly held by Mr. Jellick.

Prior to his work with the Bureau he served successively in the capacity of district engineer of the Los Angeles and San Francisco offices and as manager of the Pacific Coast offices of the Portland Cement Association.

Mr. Jellick, a registered civil engineer, has been a member of ACI since 1924.

R. C. Research Council created by Engineering Foundation

A step directed toward placing the design of reinforced concrete on a more scientific basis has been taken by the Engineering Foundation in creating a Reinforced Concrete Research Council under the chairmanship of Robert F. Blanks. The Council is collaborating closely with the Subcommittee on Ultimate Load Design of the A.S.C.E. Committee on Masonry and Reinforced Concrete. It is hoped that research projects which will soon be under way will remove some of the inconsistencies in present reinforced concrete design practices and will lead to code revisions for the design of concrete structures in accordance with the theory of ultimate design. The first project under the Council's guidance will be an investigation of columns under combined flexure and direct stress at the University of Illinois conducted by Professor F. E. Richart and his associates. Attention in these tests will be directed particularly toward the effectiveness of the compression steel as measured in the tests and as predicted by the ultimate design theory. Other projects which are being considered for the future include a study of the stress distribution in the compression zone of a reinforced concrete beam and ultimate values for shear and diagonal tension.

The following authorities (all ACI members except Messrs Garretts, Smith and Archibald) in the fields of concrete testing and design constitute the membership of the Council:

R. F. Blanks, chairman, chief research and geology div., U. S. Bureau of Reclamation; J. M. Garrelts, secretary, associate professor civil engineering, Columbia Univ.; Clyde T. Morris, professor of structural engineering, The Ohio State Univ.; F. R. Smith, asst. chief engineer, Union Railroad Co.; Raymond Archibald, I. E. Greiner Co.; Albert Cummings, Raymond Concrete Pile Co.; Harold D. Jolly, Ceco Steel Products Corp.; Arthur J. Boase*, Portland Cement Assn.; F. E. Richart, research professor of engineering materials, University of Ill.

Turner City

Turner Construction Co. has issued its annual "Turner City", a composite photograph showing buildings constructed and listing service contracts handled out of its New York, Philadelphia, Boston and Chicago offices during 1948. The

*Deceased

company carried on activities in 21 cities in 7 states. The buildings shown in the "city" involved the construction of 4,650,000 sq ft of floor space or approximately 100 acres.

During the year the Turner Construction Co., together with five other companies, under the name of River Construction Corp., carried on the construction of Lock No. 27 on the Mississippi River at Granite City, Illinois, under the direction of the Corps of Engineers, Department of the Army.

The Turner company's wholly owned subsidiary, Roane-Anderson Co., continued the management of the City of Oak Ridge, Tenn.

National Sand and Gravel Association convention

At the 33rd annual convention of the N.S.G.A. New York, Feb. 14-16, 1949, a number of papers of interest to ACI members were presented.

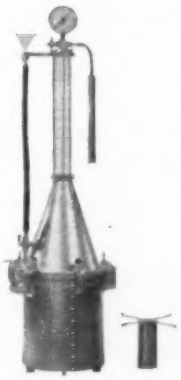
F. H. Jackson, principal engineer of tests, Public Road Administration and ACI member, presented the general aspects of "Requirements for Concrete Aggregate for Highway Construction," with certain ideas for modifications in existing practice, particularly on grading as it affects concrete uniformity. His paper was based in part on a 1947 inspection of highway construction in Germany and England.

Fred F. Bartel, asst. director of engineering of the association, summarized investigations in the association's laboratory at the Univ. of Maryland on resistance to freezing and thawing of air-entraining concrete made with questionable aggregates involving both fine and coarse aggregates over a considerable range in quality.

Corps of Engineers policies on aggregate selection of interest to local suppliers of aggregate for civil works projects were outlined by J. R. Gilbert, chief, Concrete Branch, Office, Chief of Engineers.

STEBBINS
Entrained Air Meter

Rapid
Accurate
Simple
Inexpensive
Direct Reading Percent of Air
Light Weight Compact Unit



¼ Cubic Foot Container

Stebbins Mfg. & Supply Co.
1733 Blake St. Denver, Colo.

N. R. M. C. A. meets

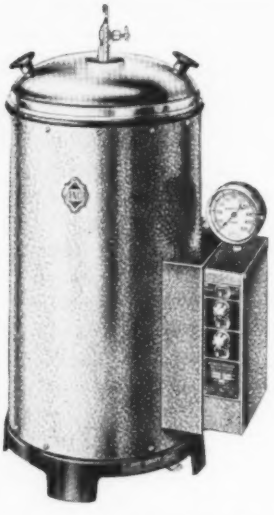
At a concurrent convention with N.S.G.A. in New York, the National Ready Mixed Concrete Assn. heard Stanton Walker, ACI past president and director of engineering of the Association, present "Recommended Practices for Sampling and Testing Ready Mixed Concrete." With Julius Warner, he proposed recommended practices for sampling and testing ready mixed concrete to determine its indicated quality.

Oliver C. Ralston and J. E. Conley, U. S. Bureau of Mines, described available types of lightweight aggregates, their characteristics and problems of production. For a summary of a Bureau publication on this subject see p. 560, Current Reviews, this issue.

Delmar L. Bloem, associate resident engineer, N.R.M.C.A., reported on investigations of various admixtures for concrete offered on the market making available limited information to member companies.

Autoclave Equipment for Accelerated Soundness Tests of Portland Cement

CENCO-MENZEL AUTOCLAVE



Accurate control of heating and cooling cycle.
Automatic pressure—temperature control.
Large, easily-read pressure gage.
High pressure chamber, chromium-plated for maximum corrosion resistance.
Built for complete safety.
Equipped with pressure safety valve.
In use in scores of cement laboratories since 1937.
Meets requirements of ASTM Standard Method C151-43.
For operation on 115 volts 50-60 cycles, also available for 25 cycles.

Write Dept. B. A. for circular 1141

CENTRAL SCIENTIFIC COMPANY

Scientific CENCO *Apparatus*

1700 IRVING PARK ROAD, CHICAGO 13

NEW YORK BOSTON SAN FRANCISCO NEWARK LOS ANGELES TORONTO MONTREAL

A. L. Alin

A. L. Alin, consulting engineer of Omaha, Nebraska, has been appointed to a part-time position as professor of hydraulic engineering at the State University of Iowa. Professor Alin will give a graduate course in the design of dams while continuing his consulting practice. He has been an ACI member since 1920.

Ammann and Whitney announce reorganization

Ammann and Whitney, Consulting Engineers, New York and Milwaukee, announce the reorganization of the firm, naming staff members Milton Brumer, Werner Ammann, Boyd G. Anderson, and James S. Whitney as associate partners. All members of the firm, with the exception of Werner Ammann, are members of ACI. Services are offered in the design and supervision of construction of industrial plants, foundations and buildings, airport facilities, special structures, bridges and tunnels, and expressways.

Committee 714 discharged

Committee 714 has been discharged with appreciation of the Board of Direction for the large amount of work it has done. Present chairman W. G. Kaiser made the recommendation for deactivation of the committee until further work is suggested in the field of concrete silos. A brief account of the activities of the committee appears on page 211, ACI Directory 1948. The committee's standard (ACI 714-46) "Recommended Practice for the Construction of Concrete Silos" will be due for re-scrutiny by a suitable committee within two years, this being a precaution taken in connection with the administration of ACI standards to avoid out-of-dateness in such documents.

Honor Roll

February 1, 1948 to January 31, 1949

The Morgans, father and son, in Illinois and Wyoming respectively, hold top honors on the Honor Roll as concluded January 31, 1949, for the year. The senior Morgan is credited with sponsoring 33 new Members in the 12 months while Newlin D. Jr. as runner up with a total of 31, was breathing hotly down pere's neck.

Raymond E. Davis with 18, and Thomas E. Stanton, 15, kept third and fourth places in California. Howard Simpson, Massachusetts, with 11 member credits made fifth place by "a nose."

Of Prof. Newlin D. Morgan, Sr., Department of Civil Engineering, University of Illinois, an ACI Member since 1928, there is more to say: his is more than a one-year record of active "member consciousness"—12 new members for the 12 months ended January 31, 1947; 19 added last year and 33 for the recently closed period, a grand total of 64 in three successive annual Honor Rolls.

Notable also in the last three years are Raymond E. Davis with a total of 54 and Thomas E. Stanton with 41 ½.

Spectacular in one year (ended January 31, 1947) was Alberto Dovali Jaime, Mexico, with 52½ (mostly students), 13 the previous year and 2½ this year for a grand total of 68.

Professor Morgan the younger didn't join ACI until 1946; started his score a year ago with 7 for a year and 31 this year; total 38.

A paper weight as memento of these accomplishments goes with ACI's appreciation to each of those mentioned in the foregoing.

And here is a record of the score of all the others on the Honor Roll ended January 31, 1949.

Newlin D. Morgan (Ill.).....33
Newlin D. Morgan, Jr. (Wyo.).....31
Raymond E. Davis (Calif.).....18
Thomas E. Stanton (Calif.).....15
Howard Simpson (Mass.).....11
James A. McCarthy (Ind.).....10½
Henry L. Kennedy (Mass.).....7½
Raphael Callejas H. (Guatemala).....7
Leon Venegas (Costa Rica).....7
Harrison F. Gonnerman (Ill.).....6
R. H. Sherlock (Mich.).....6
Oscar J. Vago (N.Y.).....6
Oliver G. Julian (Mass.).....5
Thomas C. Kavanagh (Pa.).....5
Wm. H. Thoman (Colo.).....4½
Lewis H. Tuthill (Colo.).....4½
A. Carbone (India).....4
Harold S. Carter (Utah).....4
E. I. Fiesenheiser (Ill.).....4
Myle J. Holley, Jr. (Mass.).....4
R. R. Kaufman (Ohio).....4
Jose Antonio Vila (Cuba).....4
Phil M. Ferguson (Texas).....3½
Elmo C. Higginson (Colo.).....3½
A. E. Mackney (Canada).....3½
J. Antonio Thomen (Dominican Republic).....3½
Bailey Tremper (Wash.).....3½
G. E. Troxell (Calif.).....3½
Stanton Walker (D. C.).....3½
Robert W. Freeman (Calif.).....3
J. Fruchtbaum (N.Y.).....3
Ray C. Giddings (Calif.).....3
Edward L. Howard, Jr. (Calif.).....3
John C. King (Ohio).....3
George E. Large (Ohio).....3
Leslie D. Long (N.J.).....3
F. N. Menefee (Mich.).....3
Robert B. B. Moorman (Mo.).....3
Rene S. Pulido y Morales (Cuba).....3
Robert Morris (Colo.).....3
James M. Polatty (Ga.).....3
Richard A. Roberts (Calif.).....3
Edward W. Scripture, Jr. (Ohio).....3
Thomas C. Shedd (Ill.).....3
Joseph J. Waddell (Calif.).....3
Jacob J. Creskoff (Pa.).....2½
Alberto Dovali Jaime (Mexico).....2½
Eugene Mirabelli (Mass.).....2½
Harold H. Munger (Kans.).....2½
George L. Otterson (Va.).....2½
Frank E. Richart (Ill.).....2½
John W. Robison (Colo.).....2½
John A. Ruhling (Ill.).....2½
C. H. Scholer (Kans.).....2½
John C. Sprague (Ga.).....2½
H. D. Sullivan (Wash.).....2½
Calvin T. Watts (La.).....2½
C. D. Williams (Fla.).....2½
Boyd G. Anderson (N. Y.).....2
Hugh Barnes (Calif.).....2
J. G. Bragg (Pa.).....2
F. Campus (Belgium).....2
Jose Luis Copacete (Puerto Rico).....2
John K. Class (Pa.).....2
Moses D. Cohan (N. Y.).....2
W. S. Cottingham (Wis.).....2
Walter Dardel (Switzerland).....2
W. H. Droege (Ind.).....2

Belmon U. Duvall (Ohio)..... 2
H. J. Gilkey (Iowa)..... 2
Ernst Gruenwald (N. Y.)..... 2
M. D. Hicklin (Ore.)..... 2
William E. Lumb (Canada)..... 2
F. T. Mavis (Pa.)..... 2
Hugh E. Odor (Wash.)..... 2
Walter H. Price (Colo.)..... 2
Jerome M. Raphael (Colo.)..... 2
Manuel Ray Rivero (Cuba)..... 2
Georges Ronai (Turkey)..... 2
Charles J. Sibler (N.Y.)..... 2
E. Copeland Snelgrove (England)..... 2
J. Neil Thompson (Texas)..... 2
E. W. Thorson (Colo.)..... 2
Juan Agustin Valle (Argentina)..... 2
George C. Alden (Ore.)..... 1½
Harold W. Brewer (Colo.)..... 1½
R. A. Caughey (Ia.)..... 1½
R. V. Chate (England)..... 1½
A. R. Collins (England)..... 1½
R. Torres Colondres (Puerto Rico)..... 1½
M. H. Cutler (Mass.)..... 1½
Harmer E. Davis (Calif.)..... 1½
George C. Ernst (Nebr.)..... 1½
Milton Fromer (Calif.)..... 1½
Bruce M. Johnson (Colo.)..... 1½
Francis R. McCabe (Ore.)..... 1½
Harmon S. Meissner (Colo.)..... 1½
E. J. Mueller (Wis.)..... 1½
W. E. Parker (Canada)..... 1½
Dean Peabody, Jr. (Mass.)..... 1½
Clarence Rawhouser (Colo.)..... 1½
Anton Rydland (Calif.)..... 1½
Roy T. Sessums (La.)..... 1½
James M. Shivley (Ark.)..... 1½
Harry F. Thomson (Mo.)..... 1½
John Tucker, Jr. (D. C.)..... 1½
Harold E. Wessman (Wash.)..... 1½
Ray A. Young (Wash.)..... 1½
Jerome O. Ackerman..... 1
J. B. Alexander..... 1
W. H. Allison..... 1
A. Amirikian..... 1
Edwin C. Anderson..... 1
J. A. Bakker..... 1
S. Balasubranmanyam..... 1
George E. Barnes..... 1
Paul H. Barnes..... 1
S. B. Barnes..... 1
F. Barona..... 1
Frank H. Beinhauer..... 1
Charles J. Belz..... 1
Oscar Benedetti..... 1
Rene L. Bertin..... 1
Robert F. Blanks..... 1
A. J. Boase..... 1
E. K. Borchard..... 1
Ernest M. Bowles..... 1
H. L. Bowman..... 1
William S. Bridge..... 1
Sterling Lowe Bugg..... 1
Carlos D. Bullock..... 1
H. E. Burr..... 1
John Butkus..... 1
Robert D. Chellis..... 1
C. O. Christy..... 1
Miles N. Clair..... 1
Rodger B. Collons..... 1
R. E. Copeland..... 1
Jack E. Counts..... 1
J. Slater Davidson, Jr..... 1
A. N. Dutta Choudhury..... 1
A. B. Egan..... 1
Joseph L. Fabian..... 1
T. J. Farish..... 1
A. Finlayson..... 1
T. H. Fleming..... 1
R. E. Fowle..... 1
Zorislav Franjetich..... 1
Bengt Friberg..... 1
Eric Fridh..... 1
Frank M. Fucik..... 1
Carmelo Galindez..... 1
Anthony J. Giardina..... 1
Hernando Gonzalez..... 1
George E. Goodall..... 1
R. R. Gore..... 1
Charles J. Grady..... 1
Emil A. Gramstorff..... 1
Walter Griesbach..... 1
Eugene Guillard..... 1
Luis M. Guillermety, Jr..... 1
Elliot A. Haller..... 1
J. R. G. Hanlon..... 1
Robert J. Hansen..... 1
George E. Hatch..... 1
F. E. Henman..... 1
Walter Hess..... 1
Samuel Hobbs..... 1
H. N. Howe..... 1
H. Y. Hsu..... 1
W. C. Huntington..... 1
Phil C. Huntly..... 1
Wm. R. Johnson..... 1
H. D. Jolley..... 1
W. G. Kaiser..... 1
J. E. Kalinka..... 1
Edward F. Keniston..... 1
P. J. Kennedy, Jr..... 1
M. B. Lagaard..... 1
Orla A. Larsen..... 1
Stanley D. Lash..... 1
Karl W. Lemcke..... 1
B. Leon..... 1
David V. Lewin..... 1
W. G. Limbach..... 1
Bartlett G. Long..... 1
F. A. Luber..... 1
John E. Lyons..... 1
Inge Lyse..... 1
Charles Macklin..... 1
Donald R. MacPherson..... 1
John V. Maescher..... 1
Antoni Martynowicz..... 1
Clyde P. Mason..... 1
Robert L. Mauchel..... 1
Sam McCluer..... 1
William G. McFarland..... 1
Douglas McHenry..... 1

George C. McLeod....1
F. R. McMillan....1
Carl A. Menzel....1
Jesus Mesa....1
L. J. Meszaros....1
Oliver H. Millikan....1
R. E. Mills....1
Leonard J. Mitchell....1
Hugh Montgomery....1
John J. Mullen....1
R. P. Mumford....1
T. D. Mylrea....1
Amado B. Navoa....1
William T. Neelands....1
C. H. Neill....1
N. M. Newmark....1
D. G. Norman....1
Wm. D. Nowlin....1
Calvin C. Oleson....1
J. Vicente Orozco....1
Douglas E. Parsons....1
G. S. Paxon....1
Charles W. Payne, Jr....1
R. A. Plumb....1
James J. Pollard....1
Horace A. Pratt....1
Abdul Qayum....1
R. D. Rader....1
R. C. Rao....1
Thomas J. Reading....1
J. W. Rivett....1
H. H. Roberts....1
Fernando A. Rojas B....1
E. Rosecrans....1
H. C. Ross....1
E. Gonzalez Rubio....1
C. E. Sandstedt....1
Edgar A. Sealey....1
C. Seshavatharam....1
Joseph J. Shideler....1
Joseph Sill....1
Francis P. Sloan....1
Gustave Spirz....1
John H. Stamataky....1
Willard Strode....1
W. B. Summer....1
Hale Sutherland....1
E. O. Sweetser....1
Manabu Tanaka....1
Howard M. Thomas....1
Le Roy A. Thorssen....1
T. Thorvaldson....1
Dean P. Tsagaris....1
Walter C. Voss....1
Jukka E. Vuroinen....1
W. F. Way....1
Paul Weidlinger....1
J. M. Wells....1
H. Roger Williams....1
Piers M. Williams....1
Merle D Wilson....1
Francis P. Witmer....1
K. B. Woods....1
Roderick B. Young....1

The following credits are, in each instance "50-50" with another member.

Robert W. Abbett
A. A. Anderson
Oscar Vega Arguelles
Manuel A. Arosemena
Henry Babcock
Charles Kenneth Balhiser
J. F. Barbee
A. Allan Bates
E. W. Bauman
H. W. Beckingham
Edward O. Bender
C. T. Bishop
Petrus Manzano Bonilla
P. G. Bowie
A. R. Brickler
A. S. Brock
Boyd S. Brooks
L. Rees Brooks
Byron J. Bullert
R. A. Burmeister
B. W. Burns
Jorge Carballo
Jose Luis Casas
Thomas F. Chace
Wilbur H. Chamberlin
Richard de Charms
J. E. Christiansen
James H. H. Chun
H. W. Chutter
Emil W. Colli
Harlan Coornvelt
F. Coppen
W. A. Cordon
Rolland Cravens
J. M. Crom
Albert B. Crosby
R. A. Crysler
Rembert L. Curry
E. H. Darling
Philip Davis
E. K. Davison
Charles W. De Groff
John G. Dempsey
R. F. Dierking
J. B. Dolsen
J. E. Dunn
G. J. Durant
J. Dvorak
John N. Eckle
Harlan H. Edwards
A. C. Eichenlaub
F. L. Ehasz
Arthur D. Engle
Hernan Esquivel F.
Edward E. Evans
Homer G. Farmer
William E. Feidt
E. A. Finney
Paul Fischer
Rudolph Fischl
E. M. Fleming
James Lee Ford
Frederick E. W. Fowler
Jaichand Hirachand Gandhi
Leo V. Garrity
Carl V. Gezelius
Grayson Gill
O. H. Gosswein
Howard A. Gray
J. E. Guest
Luis Alberto Truque Gurdian
Bernard Harrison
James F. Hartman
Frank W. Haws
R. G. Hennes
H. L. Henson
A. E. Hiscox
W. C. Hodgkins
John Hogan
G. W. Holcomb
John A. Houseman
Kenneth D. Hoyt
Walter L. Huber
J. W. Hubler
Manuel Castro Huerta
Robert B. Hyslop
C. L. N. Iyengar
Orville E. Jack
Frank H. Jackson
M. E. James
Bruce Jameyson
S. Friis Jespersen
E. B. Johansen
Samuel Judd
Harry D. Jumper
Anthony J. Karpinski
J. W. Kelly
Thomas A. Kelly
Thomas M. Kelly
John J. Kennedy Jr.
Frank Kerekes
Robert J. Kindl
Lane Knight
K. J. Krawczyk
H. M. Larmour
Robert E. Lee
L. G. Lenhardt
John F. Long
Jas. V. Mandia
Wallace A. Marsh
Hubert F. McDonell
Juan B. McKay
Robert T. Mercer
Erik V. Meyer
B. Michelis
Richard C. Mielenz
Alfred L. Miller
Callix E. Miller, Jr.
Ray M. Moorhead
Leo J. Muse
A. E. Myers
Israel Narrow
Henry L. Neve
Elmer H. Oechsle
Jaime Olaechea
Francisco Santos Oliva
Chester B. Palmer
Donald O. Peck
H. H. G. Pelsue
Gerald Pickett
Ben Poisner
Chesley J. Posey
Emil H. Praeger
Herman G. Protze
Warren Raeder
Raymond C. Reese
Fernando Fossas Requena
Dwight F. Roberts
John W. Roberts
Bernard L. Robinson
Fredric Rusche
Fred H. Ryan
Hilary F. Rye
Fernando Sanchez
Gustavo de Rio San Vicente
Vanus C. Sauvage
Edward B. Schicker, Jr.
Herman Schorer
J. R. Shank
Charles A. Shirk
Chester P. Siess
R. Simons
A. S. Sinclair
J. Singleton-Green
Charles M. Spofford
Edwin A. T. Spuehler
Byram W. Steele
A. R. Stuckey

Continued on p. 18

Honor Roll

Continued from p. 17

M. O. Sylliassen
Anton Tedesco
Ruth D. Terzaghi
Norman Thomas
A. G. Timms
Charles L. Townsend
Norman Wagner
Myron H. Walters
Donald R. Warren
Ray V. Warren
A. G. Watt
J. C. Watt
J. D. Whittaker
William H. Wisniski
Harry C. Witter
Charles E. Wuerpel

New Members

Continued from p. 11

Peru

Ferrand, Raul A., (St.) Av. Arequipa 501, Miraflores, Lima, Peru

Venezuela

Moros G., R. Domingo, (Indiv.) Avenida Las, Delicias No. 142, Maracaibo, Edo. Zulia, Venezuela, S. A.

Nouel C., Bernardo A., (Indiv.) Sur 7 No. 21, Caracas, Venezuela

Ali Cokyigit

Foreign employees who are bringing an international atmosphere to the staff of Gannett, Fleming, Cordry and Carpenter, consulting engineers of Harrisburg, Pennsylvania include Ali Gokyigit, University of Michigan, from Turkey.

Tools, Materials, Services

Under this heading note will be made from time to time of producer literature of presumed technical interest (and available from its source for the asking) to ACI users of tools, equipment, materials, accessories and special services.

Handbook of insulation values

A 108-page book of winter and summer insulation values lists 12,852 certified, F. H. A. approved, "U" values of various walls, floors and ceilings. The coefficients shown in the book were calculated in accordance with F. H. A. calculation procedure. Illustrating the book are 238 drawings showing typical wall, floor and ceiling sections. The drawings illustrate the proper position of the insulation to obtain stated "U" values. Most "U" values published hitherto have considered heat transfer entirely in terms of winter temperature conditions, as related to fuel savings. The new handbook includes summer "U" values because of the exceptional efficiency of reflective insulation in checking the flow of heat downward.

Typical wall sections described include masonry walls with combinations of various-dimensioned common brick, face brick, hollow tile, solid concrete and hollow concrete blocks. Included, also, are wood frame walls with common brick veneer, wood shingles, Y. P. lap siding, stucco on metal lath. The calculations cite various wall "U" values obtained from different interior finish, sheathing and insulating. Ceiling and floor sections illustrate many types of standard construction practice and widely used materials.

Copies of the new handbook may be obtained free of charge by qualified architects, engineers, and builders by writing to Silvercote Products, Inc., 161 East Erie St., Chicago 11, Illinois.

Pneumatic rubber molds for ducts in concrete

Ductube, a pneumatic rubber tubing for molding ducts in concrete, is claimed to be suitable for molding smooth-bore ducts of uniform diameter in floors, walls, columns, beams or slabs. It is said to eliminate bending and fitting of pipe or conduit. The tubes are built to withstand repeated abuse, vibrating and tamping.

Ducts may be molded in four simple operations: (1) inflate Ductube with an ordinary tire pump or compressed air line; (2) put in place in either vertical or horizontal position, either straight or curved as desired; (3) pour concrete over the inflated Ductube in the usual manner—no special pouring method is required; (4) deflate tube and pull from the molded ducts as soon as the concrete has set.

The tubes are available in standard lengths of 10, 20, 30, and 40 ft and in standard sizes for molding ducts of 1, 1½, 2, or 3 in. in diameter. Special lengths and sizes can be made to order. Lengthy runs are possible by connecting ducts in a simple manner.

Further information may be obtained from Ductube Company, 940 Shoreham Building, Washington, D. C.

Story of steelmaking

The story of modern steelmaking, from the mines to finished steel, is informally told in a new 64-page booklet, "How Steel Is Made", just issued by Inland Steel Company.

Liberally illustrated with sketches, diagrams, and photographs, "How Steel is Made" was planned primarily for the layman—executives who buy and use steel, workers who fabricate steel, engineering students, and others to whom steelmaking is of interest.

The booklet begins with a brief historical outline of steelmaking, then describes the many phases of modern steel production—mining of the essential raw materials for steel manufacture; methods of making pig iron and steel; pouring, soaking, and blooming of the ingot; and finally, rolling of the many finished and semi-finished steel products. It dwells momentarily on modern quality control and research procedures, and includes an informative exhibit showing the microstructure of various steels.

Copies of the booklet are available on request from Inland Steel Company, Room 1271, 38 S. Dearborn St., Chicago 3, Illinois.

Tilting trailers

Two new inexpensive tilting trailers, designed to speed job-to-job hauling of tractors, mixers, compressors, and similar bulky loads, have been announced by La Crosse Trailer Corp., La Crosse, Wisconsin. The manufacturer reports both trailers can easily be loaded or unloaded by one man without skids or blocks, and can be used behind any standard ½ to 1-ton truck equipped with standard pintle hook. The tandem axle 4 wheel model has a capacity of 6 to 8 tons while the single axle dual-tire model is rated at 8 to 10 tons.

Modular building units

A lightweight precast concrete called Durisol is made of chemically-mineralized wood shavings combined with portland cement and molded into modular slabs, blocks and tiles to meet a variety of uses in building construction. Units are lightweight for easy handling, rigid for semi-structural applications, fire resistant, insulating and sound-absorbing. The material is said to be free from attack by mold, fungus, rats or termites and to be unaffected by moisture. It is non-caustic and may be installed in direct contact with metal structures.

To provide complete modular flexibility, Durisol units are made in a range of sizes and types that meet standard needs and reduce on-the-site labor. It can also be cut with a saw (hand or power) when non-modular units are required. It is easily fastened in place with nails, screws, clips or mortar. Units are available for a wide variety of uses such as: sheathing, stucco base, plaster base, partitions, ceilings, sub-flooring, and roof slabs. Special units are factory-coated with waterproof white portland cement for use as insulating veneer or as an entire exterior or interior wall with minimum framework.

Further information on this material is available from Durisol, Inc., 420 Lexington Avenue, New York 17, New York.

Flexible template

The Flexible Template is said to duplicate curves full size in a matter of seconds, often saving hours of curve plotting, tedious drawing and template making. It can be set and locked to any desired shape, contour, curve or radius and transferred anywhere for construction or checking purposes. Springs back to original position when unlocked, ready for re-use. Used to copy curves from blueprints, to take off construction lines, and in many other ways.

Information may be obtained from C. H. Clark and W. E. Poggenburg, 4900 Wynnefield Avenue, Philadelphia, Pa.

Mechanical bond between concrete and plaster

Kifs are devices which may be attached to forms for concrete, claimed to be useful not only for forming a keyed plastering surface but for providing anchors for suspending overhead sprinkler, water and heater pipes and ducts, as well as for anchoring masonry veneers.

The Kifs, small elastic button-like nobs the size of a large daisy, are nailed about 6 in. apart to the wooden forms which support the concrete when it is being poured. They may be walked on and worked over without damage or interference to other trades. After the concrete has hardened, the removal of the forms pulls the Kifs out, leaving undercut, cone-shaped cavities. When applied, the plaster then squeezes into the cavities forming a keyed, permanent, mechanical bond between the concrete and the plaster.

The Kifs remain in place on the forms ready for the re-use without further preparation. After the job is complete, the Kifs are removed from the forms by hand.

Kifs may be attached to all sides of beams and column forms for preparing all surfaces for plastering. In climates where precast, all-concrete homes are practical, they may be attached to the horizontal wood forms and may be applied to wood "floats" laid on top of the freshly poured concrete. The undercut niches may be formed on both sides of the wall. Plaster then can be applied to the interior walls, while the exterior is ready for a direct application of stucco or other commonly used exterior veneers.

A substantial saving is claimed for this method over the use of metal lath.

Special corrugated or perforated metal anchors have been designed for anchoring brick work, marble, slate or terra cotta to concrete walls and spandrels.

Buffalo Products, Inc., manufacturers of Kifs, is a subsidiary of Frontier Industries, Inc., located at 315 Babcock Street, Buffalo, N. Y.

NOTICE!

Copies of original books and magazine articles reviewed in the Current Reviews section of the JOURNAL are **not** available from ACI. The footnote formerly published with that section indicated that separate prints of *the reviews section itself* are available, not separate prints of the individual articles reviewed, as some readers have assumed.

SYNOPSES of recent ACI Papers and Reports

Institute papers of this JOURNAL Volume 20 are currently available at prices indicated. Please order by title and title number.

RECOMMENDED PRACTICE FOR WINTER CONCRETING METHODS (ACI 604-48).....45-1

Price 50 cents.

REPORT of COMMITTEE 604—Sept. 1948, pp. 1-20, (V. 45) in special covers

Supersedes 44-13.

This ACI standard establishes methods of cold-weather concreting for thin sections and mass concrete. Heating of materials, accelerators and anti-freezes, curing and temperature records during curing, subgrade (or base) preparation, protective coverings during curing, and form removal are discussed for both types of job, and preferred methods are indicated. An appendix entry outlines objectives of the special winter methods with background material which indicates the "why" of some of the recommended practices. Charts in the appendix indicate effect of curing temperature on concrete strength, and a list of 135 selected references to periodical literature on winter concreting methods is included.

RELATIVE STRENGTHS OF PORTLAND CEMENT MORTAR IN BENDING UNDER VARIOUS LOADING CONDITIONS.....45-2

Price 35 cents.

JACOB PORTER FRANKEL—Sept. 1948, pp. 21-32 (V. 45)

Assuming the behavior of standard mortar to be similar to that of plain concrete, tests were performed on 99 small mortar beams under sixth-, third- and center-point loading to verify the applicability of the statistical theory of the strength of brittle materials to concrete structures. The theory, as first developed by Weibull, is briefly analyzed, compared to the methods used by Tucker, and finally applied to the tests at hand. Agreement between the theoretical and experimental findings is so close as to warrant future testing on plain concrete specimens.

TERRAZZO AS AFFECTED BY CLEANING MATERIALS.....45-3

Price 35 cents.

D. W. KESSLER—Sept. 1948, pp. 33-40 (V. 45)

Effect of cleaning materials on terrazzo was studied using solutions of the following detergents: soda ash, trisodium phosphate and synthetic sulfonate. Seventy-six different aggregates were used in preparing terrazzo disks which were moist cured 3 months and then surface ground to form a shallow dish. Detergent was allowed to stand in dish 30 minutes before rinsing and drying (at 105 C). This testing cycle was repeated until deterioration tendencies were established. Soda ash solution proved much more destructive than the trisodium phosphate; however, up to the time of writing (after 850 cycles) the synthetic sulfonate had caused no failure.

TRANSPORTING READY-MIXED CONCRETE IN OPEN DUMP TRUCKS.....45-4

Price 35 cents.

R. A. BURMEISTER—Sept. 1948, pp. 41-56 (V. 45).

Milwaukee experience in hauling air-entraining concrete in open dump truck bodies is recorded. Slump, air content, specific weight, compressive strength and workability tests were made on concrete before and after a trial run over bumpy pavement. Some similar tests were made on concrete being placed in a Milwaukee street after open truck transportation. The author concludes that open trucks for hauling ready-mixed air-entraining concrete are satisfactory when air content is between 3 and 6 percent (5 to 6 sacks cement per cu yd) and hauling time is 45 minutes or less, provided that aggregate moisture, grading and slump are carefully controlled.

A STUDY OF ALKALI-AGGREGATE REACTIVITY BY MEANS OF MORTAR BAR EXPANSIONS.....45-5

Price 35 cents.

T. M. KELLY, L. SCHUMAN and F. B. HORNIBROOK—Sept. 1948, pp. 57—80 (V. 45)

Many types of aggregate were combined in varying amounts and sizes with high- and low-alkali cements and formed into 1 x 1 x 10-in. mortar bars. The bars were stored either at 70 F or at 100 F and their expansions measured at ages ranging from 1 month to 4 years.

In combination with high-alkali cements, opal, opaline chert and a siliceous dolomitic limestone were found to cause greatest expansion. Certain aggregates containing volcanic glasses and some natural sands and gravel also caused excessive expansion; with one exception, these sands contained small amounts of opal.

Greatly delayed expansion resulted with the very fine sizes of opal, particularly in combination with high-soda cement. Similar behavior resulted with minus No. 81 size opal and low-alkali cement with either Na_2SO_4 or K_2SO_4 additions.

Materials such as dehydrated kaolin, soda feldspar, magnesium fluosilicate, acetic acid and calcium hydroxide added in small amounts as correctives were ineffective. However, diatomaceous earth in sufficient quantity as a cement replacement eliminated expansion.

REINFORCED CONCRETE WALL AND COLUMN FOOTINGS PART I.....45-6a

Price 60 cents.

FRANK E. RICHART—Oct. 1948, pp. 97-128 (V. 45)

PART II.....45-6b

Price 35 cents.

FRANK E. RICHART—Nov. 1948, pp. 237-260 (V. 45)

This paper presents, in two parts, a report on an experimental investigation of reinforced concrete footings. It represents the first extensive study of the subject since the well-known work of Talbot in 1913. In the meantime there have been many developments in materials, as well as in design methods.

Important developments from these tests include the following: (a) Definite proof that the tensile and bond resistance depend upon the bending moment and shear found by statics by consideration of the full applied load, and not 85 percent thereof, as assumed in current building codes; (b) Hooked ends of bars showed no particular advantage in bond resistance over straight bars, particularly when deformed bars of new improved types were used; (c) Hooked bars produced very little effect on resistance to diagonal tension, much less than is commonly assumed; (d) Welded mats proved particularly effective in resisting end slip of bars; (e) Footing caps or piers are effective in reducing the amount of reinforcement and in increasing the load capacity of footings; and (f) Diagonal tension seems to be the point of weakness in current design practice. The factor of safety of thin footings in this respect appears greater than in thick footings, and is generally greater in rectangular than in square footings when the conventional methods of computation are used.

In these tests, major emphasis has been placed on the isolated column footing. Principal attention has been

given to the resistance of footings to failure by bond, diagonal tension and tension in the steel. Test footings were designed to produce these various types of failure. In addition, studies were made of the behavior of rectangular footings and footings having intermediate capitals or piers.

The tests featured measurements of tensile and compressive strains, deflections and slippage of bars. In some of the tests three grades of concrete were used, in others, five types of reinforcing bar were employed. In most of the tests, the footings were 7 ft square. They were supported on a bed of steel springs and tested in a large testing machine.

This paper merely attempts to present and interpret the test results, and does not offer recommendations as to possible changes in design procedures at this time.

LATTICE ANALOGY IN CONCRETE DESIGN.....45-7

Price 35 cents.

DOUGLAS McHENRY—Oct. 1948, pp. 129-140 (V. 45)

The lattice analogy is a scheme for solving two-dimensional stress problems in which the involved mathematical methods of the theory of elasticity are replaced by simple computations. The solid section is replaced by an equivalent lattice or framework which may be solved by methods applicable to indeterminate structures. Stress distribution in sections of complex shape and with complex loading may be determined by successive approximations which involve only substitution in simple equations. The method is illustrated by application to the problem of stresses in a deep beam with off-center loading.

STRENGTH OF PRECAST CONCRETE FLOOR JOISTS.....45-8

Pricd 35 cents.

JACOB FELD—Oct. 1948, pp. 141-148 (V. 45)

The strength and usability of precast concrete floor joists left in the open without any protection for a year, when a housing project was abandoned, were proved by load test on a slab section incorporating the poorest joists. This paper describes the condition of the joists, the load test results and indicates that a greater tolerance can be safely permitted in the visible defects of precast concrete joists.

EFFECT OF ENTRAINED AIR ON CONCRETES MADE WITH SO-CALLED "SAND-GRAVEL" AGGREGATES.....45-9

Price 35 cents.

PAUL KLIEGER—Oct. 1948, pp. 149-164 (V. 45)

Sand-gravel aggregates (maximum particle size 3/8 in.) used with non-air-entraining cement produce concretes containing from 3 to 4 percent air. The same mixes made with air-entraining cement and the sand-gravel aggregates have air contents from 8 to 13 percent. Effect of this larger amount of entrained air is given in terms of tests of flexural and compressive strength, freezing and thawing resistance, and length changes in varying storage conditions. Effect of "sweetening" the sand-gravel by addition of 30 percent of coarser aggregate is noted.

BURNED SHALE AND EXPANDED SLAG CONCRETES WITH AND WITHOUT AIR-ENTRAINING ADMIXTURE.....45-10

Price 35 cents.

P. H. PETERSEN—Oct. 1948, pp. 165-176 (V. 45)

The physical properties of several lightweight aggregate portland cement concretes made with burned shale or expanded slag were investigated at the National Bureau of Standards. Three grades of concrete were made with each aggregate. Air-entrainment greater than 20 percent is reported for the mixtures leanest in cement, an air-entraining admixture being used to increase the workability of all but the richest concretes. Compressive, flexural and bond strength data are given as well as resistance to heat transfer, rain penetration and water penetration by capillarity. Also included are the coefficients of thermal expansion, shrinkage, and values for change in length due to wetting and drying.

CEMENSTONE PRECAST CONSTRUCTION.....45-11

Price 35 cents.

A. C. GRAFFLIN—Nov. 1948, pp. 193-204 (V. 45)

Standardized precast reinforced concrete structural members and panels point toward economy in fireproof construction of ordinary buildings up to four stories. The method described has been so developed that architects and engineers can select from tables reinforced concrete beams, columns, roof, floor, and wall panels in the same manner as for structural steel. Design is in accordance with the ACI Code and costs compare favorably with similar structures framed of steel and fireproofed with concrete.

SAND GRADING INFLUENCE ON AIR ENTRAINMENT IN CONCRETE.45-12

Price 35 cents.

M. A. CRAVEN—Nov. 1948, pp. 205-216 (V. 45)

Four series of mixes with varying air-entraining agents, cement factors, and sand grading and content were prepared in order to observe sand grading effect on concrete. Graphic record is presented of air content and W/C plotted against fineness modulus of sand; flow and compressive strength plotted against fineness modulus of sand; and air content plotted against percent of No. 30—No. 50 sand and total sand percentage. Generally the percentage of air entrained in concrete increased with decrease in fineness modulus of sand. Quantity of air appears to be a function of the quantity of No. 30—No. 50 sand. Effect of grading and quantity of sand on other properties of fresh and hardened concrete is noted.

INFLUENCE OF SIZE GRADING OF SAND ON AIR ENTRAINMENT.45-13

Price 35 cents.

E. W. SCRIPTURE, Jr., F. B. HORNIBROOK, and D. E. BRYANT—Nov. 1948, pp. 217-228 (V. 45)

Field reports indicated difficulty with certain sands in securing the desired amount of entrained air; this was frequently attributed to the size grading of the sand, particularly to a deficiency in the finer fractions. An experimental investigation was undertaken in view of the paucity of published data on this subject. Mixes of sand and water alone, 1:4 and 1:2 mortars, and concrete mixes were made with and without air-entraining agents and the air contents determined. While size grading of the sand had a very large influence on air entrainment in a mixture of sand and water alone, this effect was smaller in mortars and very small in concrete mixes. It was found that maximum air was entrained by the 28–48 mesh size sand rather than the 48–100 mesh size.

USE OF PREPACKED AGGREGATE CONCRETE IN MAJOR DAM CONSTRUCTION.....45-14

Price 35 cents.

PAUL BAUMANN—Nov. 1948, pp. 229-236 (V. 45)

This paper sets forth the method of construction of a major gravity dam whereby the average temperature of the mass concrete may be controlled within the mean annual and the dry shrinkage reduced to a fraction of that of conventional concrete by means of prepacking of the coarse aggregate to a density corresponding to 30 percent voids; precooling the aggregate; the intrusion of the coarse aggregate by mortar with a cement content of two sacks per cu yd or less; and the internal post-cooling, curing and draining of the mass concrete. (This process is known as the Prepakt method of construction.)

RATIONAL ANALYSIS AND DESIGN OF TWO-WAY CONCRETE SLABS.....45-15

Price 60 cents.

C. P. SIESS and N. M. NEWMARK—Dec. 1948, pp. 272-316 (V. 45)

A new method for the design of two-way building slabs is proposed and its step-by-step development is described. A new moment distribution procedure is used to compute moments in a number of rectangular slabs continuous over rigid beams. Several variables are studied in the analyses and include: the ratio of sides, the effect of discontinuous edges, the torsional stiffness of the beam, various types of loading, and combinations of panels of various sizes and shapes. Certain conclusions are stated regarding the types of loading to be considered and the values of beam torsional stiffness to be assumed in the development of the design procedure.

Moments obtained in the foregoing analyses are modified to take into account the effect of additional variables. First, the slab moments are increased by various amounts as a result of the deflection of the beams. Next, separate coefficients for dead and live load are replaced by combined coefficients for a live load-dead load ratio of 3.0. Then follows a study to determine the effect of discontinuous edges, and this variable is eliminated from the procedure. Finally all slab moments are reduced by 20 percent in recognition of the redistribution of moments at high load. Additional studies are then made of the distribution of moments across the width of the slab and of the moments to be used for the design of the beams to complete the development of the design procedure.

A proposed design procedure is presented in specification form and is compared with other procedures of a similar nature and with the moment distribution procedure described in the appendix.

CORROSION RESISTANCE TESTS OF CONCRETE FLOORS—WITH AND WITHOUT METALLIC AGGREGATE.....45-16

Price 35 cents.

E. W. SCRIPTURE, JR. and C. H. SAKRYD—Dec. 1948 pp. 317-324 (V. 45)

An attempt has been made to determine the conditions under which concrete floors can be expected to give good service. Resistance to abrasion was determined on specimens with and without metallic aggregate after exposure to typical reagents selected from the groups described by the Joint Committee as having corrosive effects of an intermediate nature between no attack and severe attack. On the basis of the experimental results the reagents in the Joint Committee report have been classified in relation to anticipated service of concrete floors.

FLEXICORE PRECAST FLOOR AND ROOF SLABS.....45-17

Price 35 cents.

GAYLE B. PRICE—Dec. 1948, pp. 325-340 (V. 45).

This paper describes briefly the design, manufacture, erection procedures and use of Flexicore slabs in precast concrete floor and roof systems. The Flexicore slab has a standard cross section 12 in. wide and 6 in. thick, lightened by two holes 4⅝ in. in diameter running the full length of the slab which is made in inch variations of length up to a maximum of 22 ft 6 in. Reinforcement consists of two bars in the top of the slab for "handling" stresses and two or three prestressed tension bars at the bottom to take care of bending moment.

PROPOSED RECOMMENDED PRACTICE FOR THE APPLICATION OF PORTLAND CEMENT PAINT TO CONCRETE SURFACES.....45-18

Price 35 cents.

REPORT of COMMITTEE 616—Jan. 1949, pp. 353-368, (V. 45)

This report is a studied revision by a reorganized committee of the previous report of Committee 616 published in June 1942, based upon criticisms of that report plus information from more recent tests and experiences. Included are recommendations for appropriate usage, age of concrete, preparation of surface, and the preparation, application and curing of portland cement paint. Three appendixes discuss composition, manufacture and storage and, general characteristics and factors affecting durability.

DRY MORTAR AS A BEARING AND GROUTING MATERIAL.....45-19

Price 35 cents.

BOYD S. BROOKS—Jan. 1949, pp. 369-380, (V. 45)

In 1939, at the navy's new David Taylor Model Basin at Carderock, Md., there was an urgent need for a non-shrinking bearing material to be used under the cast-steel "chairs" which support the rails on which the towing carriages run. Requests for information concerning materials and installation procedures produced such varied answers that investigation was started to find the best material and the best procedures for placing it.

The bearing material selected for use was 1:3 portland cement mortar, with 4.13 gallons of water per sack of cement. This was rammed firmly into place.

This bearing material has now been in place over a period of about 9 years under extremely severe conditions and without exception has proved to be excellent in function and condition.

DIRECT DIMENSIONING OF RECTANGULAR SECTIONS.....45-20

Price 35 cents.

MICHEL BAKHOUM—Jan. 1949, pp. 381-396 (V. 45)

On the basis of the standard theory for reinforced concrete design, methods have been derived for the direct dimensioning of rectangular sections, with or without compression reinforcement, when subjected to simple bending, eccentric compression or eccentric tension. Two cases are considered: when the depth of the section is fixed and when it is not fixed. The conditions leading to minimum reinforcement are also studied. The solutions developed have been simplified by the use of curves which are applicable for pure bending as well as for eccentric compressive or tensile forces and cover any values of f_c, f_s and n. The curves allow making a variety of designs in a relatively short time and thus help in choosing the most economical or most suitable section.

STRENGTHENING BRIDGE SLABS WITH GROUTED REINFORCEMENT.45-21

Price 35 cents.

S. O. ASPLUND—Jan. 1949, pp. 397-408

On the project described the negative reinforcing bars settled as much as 2½ in. out of correct position. Various means for incorporating negative reinforcement at correct height in the finished structure are discussed. The method selected of grouting additional bars in grooves cut with the aid of diamond saw is described. Tests on beam specimens made with some bars encased in the concrete and some bars grouted into diamond-sawed grooves, give results identical with those to be expected for all bars normally encased; ultimate strengths conform closely to the plastic bending capacity of reinforced concrete according to design methods originated by C. S. Whitney.

COMPARATIVE DESIGNS OF A SEGMENTAL SKEWED FRAME CONCRETE BRIDGE BY THE STRAIGHT LINE AND PLASTIC THEORY METHODS.....45-22

Price 35 cents.

MILTON BRUMER—Jan. 1949, pp. 409-420 (V. 45)

The designs of a segmental skewed frame concrete bridge are discussed and compared as developed by both the straight line and plastic theory methods. The plastic theory will lead to rigid concrete frames of more economical proportions capable of sustaining equal or greater loads than frames designed by the straight line method. Effects of volume changes and yielding supports are also of smaller consequence. Deflections may be expected to be greater. The author concludes that further substantiation of the plastic theory is desirable.

SOME FACTORS AFFECTING AIR ENTRAINMENT.....45-23

Price 35 cents.

E. W. SCRIPTURE, Jr. and F. J. LITWINOWICZ—Feb. 1949, pp. 433-444 (V. 45).

A program was undertaken to investigate the influence of various factors on the amount of air entrained in concrete mixes with and without air-entraining agent added. The variables so far studied are slump, cement factor, and sand-total aggregate ratio. The results indicate that the two latter factors are of major importance but that slump has less effect. In general, entrained air content increases with increasing slump, decreases with increasing cement factor, sand-total aggregate ratio.

EXPERIMENTAL AIDS IN STRUCTURAL CONCRETE DESIGN...45-24

Price 35 cents.

R. E. GLOVER, O. J. OLSEN and CARL ZANGAR—Feb. 1949, pp. 445-468 (V. 45).

Statically determinate and indeterminate structures are generally designed by analytical methods. This paper describes several experimental aids which have been used in the design and stress analysis of statically indeterminate structures at the laboratories of the Bureau of Reclamation. These methods are used where analytical procedures cannot conveniently be applied. Described herein are photoelastic methods using both the polariscope and interferometer, the Beggs Deformeter, Stresscoat and electric SR-4 strain gages. Applications are given for each method.

PRACTICES, EXPERIENCES, AND TESTS WITH AIR-ENTRAINING AGENTS IN MAKING DURABLE CONCRETE.....45-25

Price 35 cents.

R. F. BLANKS and W. A. CORDON—Feb. 1949, pp. 469-488 (V. 45)

The authors discuss the advantages of air entrainment with regard to durability, permeability, workability, reduction in alkali expansion, time saving, reduction in water and cement, temperature rise, strength, abrasion resistance, monolithic lightweight concrete and mass concrete. U. S. Bureau of Reclamation experiences are discussed with regard to pumping, transportation and placing, grading of aggregates, factors affecting the amount of air entrained Air-entraining agents and air-entraining cements are discussed briefly. There is a brief section on the measurements of entrained air in concrete with a recommended procedure.

CONCRETE FLOOR FINISHING.....45-26

Price 35 cents.

GERALD MILSOM—Feb. 1949, pp. 489-492 (V. 45).

A concise discussion of the requirements for durable concrete floors including careful selection and control of materials and attention to weather conditions. The importance of proper equipment, working conditions and curing of the finished concrete are emphasized.

PLASTIC FLOW OF CONCRETE AT HIGH OVERLOAD.....45-27

Price 35 cents.

J. R. SHANK—Feb. 1949, pp. 493-500 (V. 45).

New data on the plastic flow of concrete prompted a reconsideration of the assumption that plastic flow is directly proportional to the unit stress applied. Tests are described in which deformations of 6 x 12-in. concrete cylinders were measured while under sustained loads approaching the ultimate strength. Plotted data are shown to demonstrate that the change in the rate of plastic flow appears to be uniform up to three-quarters or more of the ultimate strength with an abrupt change at the "true" ultimate strength. Included are other data indicating that the plastic flow curve has the same general form independent of time and applied load leading to a new consideration in expressing factor of safety and developing beam theories on a factor of safety basis.

CONTROL OF CONCRETE PAVEMENT SCALING CAUSED BY CHLORIDE SALTS.....45-28

Price 35 cents.

B. D. TALLAMY—Mar. 1949, pp. 513-520 (V. 45)

Under modern traffic requirements on heavily traveled roads salt-treated abrasives will not remove ice quickly enough. As maintenance forces have struggled to meet the demand for uninterrupted service the use of straight salts has become increasingly common. Direct applications of up to 600 lb of salt per two-lane mile are required to combat extreme icing conditions. Pavements constructed to withstand the weak brines deposited by salt-treated abrasives cannot stand up under straight salt. New pavements appear more vulnerable than those two or more years old.

The observed resistance to salt action of the oil soaked center streak of uphill traffic lanes led New York state highway engineers to investigate the feasibility of the use of dilute oil applications as a protective measure. Laboratory tests indicate successful results may be expected. In the late summer of 1948 oil spray equipment was developed in time to treat nearly 60 miles of new concrete highways, which should provide a wide base for field observation of the effectveness of the method.

EXPERIENCE WITH AIR-ENTRAINING CONCRETE IN NEW JERSEY...45-29

Price 35 cents.

CHARLES M. NOBLE—Mar. 1949, pp. 521-528 (V. 45)

This paper outlines experience in New Jersey with air-entraining concrete utilized primarily to combat attack by de-icing chemicals. Damage to concrete pavements in New Jersey usually is associated with heavy loads. Freedom from disintegration troubles, except in isolated cases, is attributed to excellent aggregates, rigid laboratory control, mix design, inspection of material and plant equipment and construction supervision at the job site.

Air-entraining portland cement concrete was first specified for a concrete pavement contract in 1945 and since then has been used on many contracts with notable success to prevent deterioration due to scaling. The same high standards for materials, mixture design, laboratory control, field inspection and attention to details must be maintained as with ordinary concrete. Pavements thus far constructed in New Jersey with air entrainment have not

scaled or shown any signs of disintegration when ice control chemicals have been properly mixed with abrasives at a rate not exceeding 75 lb per cu yd of abrasive. It is too early to form judgment but indications are the results achieved fully justify the use of air-entraining concrete.

DESIGN DETAILS FOR ARCHITECTURAL CONCRETE.....45-30

Price 35 cents.

J. J. HOGAN—Mar. 1949, pp. 529-540 (V. 45)

"To achieve beauty with concrete, as with any other material, requires studied design and skillful craftsmanship" are the words of architect Robert D. Murray. This paper directs attention to the matter of "studied design". The skilled craftsmanship required to construct architectural concrete must be preceded by proper design by the architect and engineer. The designer must understand and appreciate the fundamental requirements of this type of construction. This paper discusses considerations appropriate to achieving the desired texture of surface as affected by form materials selected. Ornamentation should be considered in every detail from the standpoint of practicability. Construction joints are inevitable and should be specified in logical locations. Similarly, control joints should be specified with an understanding of where the most likely cracks due to volume change will occur and the design of reinforcement should be coordinated with the location of the control joints. Numerous illustrations support the text.

CONSTRUCTION PRACTICES FOR ARCHITECTURAL CONCRETE.....45-31

Price 35 cents.

E. B. OBERLY—Mar. 1949, pp. 541-552 (V. 45)

Detailed plans and carefully drawn specifications executed by a contractor and superintendent with the know-how of form construction are necessary for good architectural concrete. This paper describes some of the more important construction practices to be considered to achieve the results desired. Form details and concrete quality are discussed in considerable detail, the former being supported by illustrations.

The AMERICAN CONCRETE INSTITUTE

is a non-profit, non-partisan organization of engineers, scientists, builders, manufacturers and representatives of industries associated in their technical interest with the field of concrete, dedicated to the public service. Its primary objective is to assist its members and the engineering profession generally, by gathering and disseminating information about the properties and applications of concrete and reinforced concrete and their constituent materials.

Since 1905 that primary objective has been achieved by the combined membership effort. Individually and through committees, and with the cooperation of many public and private agencies, members have correlated the results of research, from both field and laboratory, and of practices in design, construction and manufacture.

The work of the Institute has become available to the engineering profession in annual volumes of ACI Proceedings since 1905. Beginning 1929 the Proceedings have first appeared periodically in the Journal of the American Concrete Institute and in many separate publications.

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Relative Strengths of Portland Cement Mortar in Bending Under Various Loading Conditions—Jacob Porter Frankel

Terrazzo as Affected by Cleaning Materials—D. W. Kessler

Transporting Ready-Mixed Concrete in Open Dump Trucks—R. A. Burmeister

A Study of Alkali-Aggregate Reactivity by Means of Mortar Bar Expansions—T. M. Kelly, L. Schuman, F. B. Hornibrook

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ACI publications in large current demand

Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-48)

Approved as a Standard of the American Concrete Institute at its 44th Annual Convention February 25, 1948. Ratified by letter ballot June 29, 1948. It is of large format, bound to lie flat and presents typical engineering and placing drawings with discussion calling attention to important considerations in designing practice. It was prepared to simplify, speed, and effect standardization in detailing. It is believed to be the only publication of its kind in English. It is meeting wide acclaim among designers, draftsmen and in engineering schools. Price—$3.00, to ACI Members—$1.75.

ACI Standards—1948

202 pages, 6x9 reprinting current ACI Standards: Building Code Requirements for Reinforced Concrete (ACI 318-47); Minimum Standard Requirements for Precast Concrete Floor Units (ACI 711-46); five recommended practices: Use of Metal Supports for Reinforcement (ACI 319-42); Measuring, Mixing and Placing Concrete (ACI 614-42); Design of Concrete Mixes (ACI 613-44); Construction of Concrete Farm Silos (ACI 714-46); Winter Concreting Methods (ACI 604-48); and two specifications: Concrete Pavements and Bases (ACI 617-44) and Cast Stone (ACI 704-44)—nine ACI Standards in one book, $2.00 per copy—to ACI Members, $1.25. The "Detailing Manual" (ACI 315-48) is available separately only at its established price—see above.

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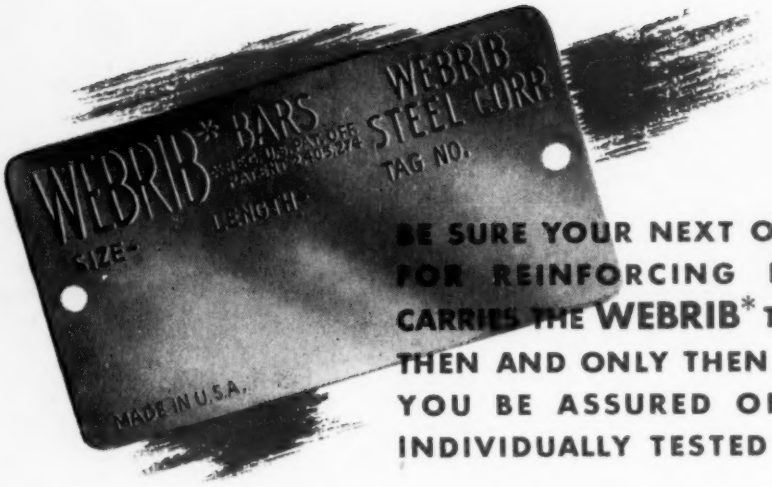

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Vol. 20 April 1949 No. 8

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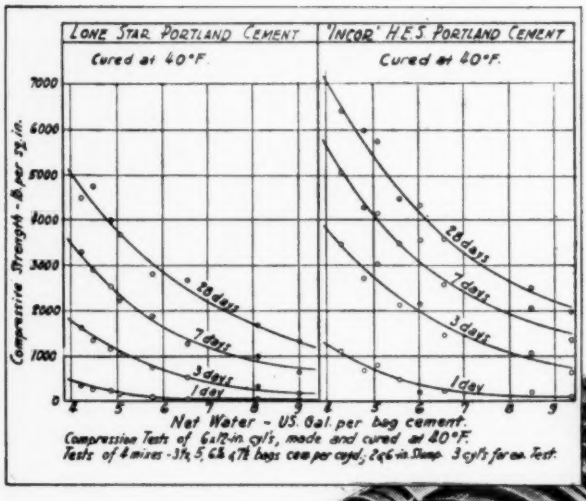

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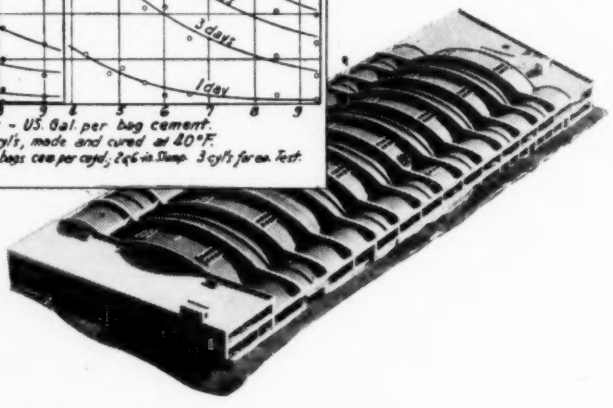

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Vol. 20—No. 8 April 1949 (Proceedings Vol. 45)

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

Published by the American Concrete Institute under the authority of its Board of Direction

ACI history and objectives. The Institute was founded 1905, incorporated in the District of Columbia in 1906 as The National Association of Cement Users, the name changed 1913 by charter amendment. It was re-incorporated August 8, 1945 with these objects: "to organize the efforts of its members for a non-profit public service in gathering, correlating and disseminating information for the improvement of the design, construction, manufacture, use and maintenance of concrete products and structures.

Papers, committee reports and other contributions to the **Proceedings** pages of this Journal, as submitted in furtherance of Institute objectives, are, if published, accepted on the sole responsibility of their authors. Institute authority attaches only to standards adopted as provided in the By-Laws. All manuscripts prepared with a view to Journal publication should be submitted in triplicate, addressed: Secretary, Technical Activities Committee, American Concrete Institute, New Center Building, Detroit 2, Michigan.

Advertising—See statement under "Alphabetical List of Advertisers", final page **News Letter** section.

Journal subscription price $10.00 per year payable in advance. Members receive the Journal as one of the considerations of dues payment. The Journal is issued monthly, January to June and September to December (the volume year September to June being completed by title page, contents, indexes and closing discussion in Part 2 of the subsequent December issue).

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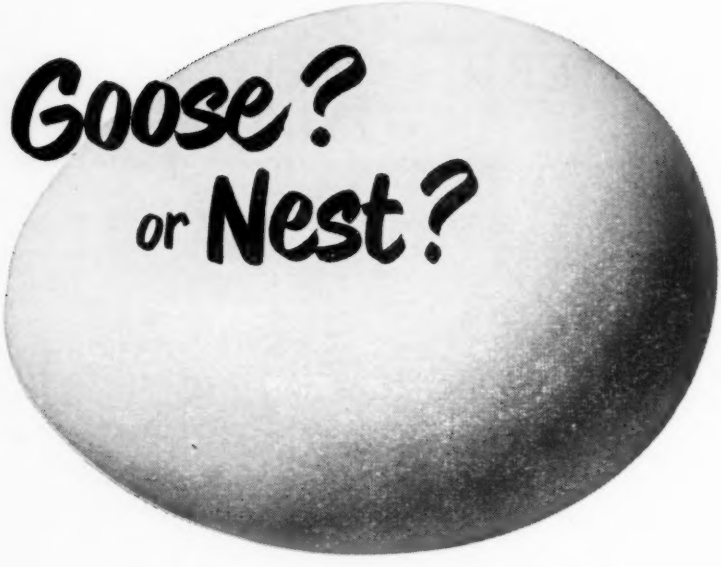

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POLICY

It is the policy of the American Concrete Institute to encourage participation by its members and others in the work of extending the knowledge of concrete and reinforced concrete as a basis for improved products and structures.

To this end the Board of Direction has assigned to the Technical Activities Committee the responsibility of selecting for publication such papers, committee reports, discussions and other contributions or parts of such contributions as, in the judgment of the Committee, seem to offer most of value in attaining Institute objectives within space requirements, consistent with budget limitations.

See News Letter p. 3 for the story of the 1949 convention. Each paper presented is described briefly and discussion is summarized. Notes on committee meetings are also of interest.

"Tests of Lightweight-Aggregate Concrete Designed for Monolithic Construction," p. 581, summarizes a U. S. Bureau of Reclamation study of lightweight aggregates found in western United States. The tests were made for the Housing and Home Finance Agency to provide basic data for engineers and architects working with these materials. A parallel project by the National Bureau of Standards on eastern lightweight aggregates will be reported in "Lightweight-Aggregate Concrete," scheduled for the May JOURNAL. Together the two papers furnish information on the physical properties of common American lightweight mineral aggregates.

Healthy ACI membership growth continues. Without the stimulus of a drive, the first three months of this year showed applications received as follows: January 126, February 126, and March 122—a remarkably consistant showing. The membership total as of April 1 stands at 4667.

A third printing again makes available "Manual of Standard Practice for Detailing Reinforced Concrete Structures." Price—$3.00, to ACI members $1.75.

Manuscripts of papers, discussions and reports should be sent in triplicate to:

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ACI publications in large current demand

Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-48)

Approved as a Standard of the American Concrete Institute at its 44th Annual Convention February 25, 1948. Ratified by letter ballot June 29, 1948. It is of large format, bound to lie flat and presents typical engineering and placing drawings with discussion calling attention to important considerations in designing practice. It was prepared to simplify, speed, and effect standardization in detailing. It is believed to be the only publication of its kind in English. It is meeting wide acclaim among designers, draftsmen and in engineering schools. Price—$3.00, to ACI Members—$1.75.

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Retiring President Blank's "remarks" are refreshing in their imaginative conjecture.

Concrete of the Future*

By ROBERT F. BLANKS†

SYNOPSIS

Mr. Blanks, in his remarks as retiring president, depicts what concrete might be 50 years in the future and discusses the role of the engineer in that era.

I notice the agenda for this afternoon has another mistake on it. The next item listed is an "address" by the President. Those of you who know me know that I never made a speech in my life, and an "address" sounds even more formidable than a speech. All I am going to do is to talk a little bit and sit down. I won't take up much of your time because I am mindful, and was reminded just this morning, of Clint Jansen's famous definition of an engineer. Clint Jansen says an engineer, when he is on his feet, is an ass, and when he is on his ass, is a genius. So I intend to set it down very quickly.

Concrete has been used to make almost everything under the sun, so I conjured an idea: Why wouldn't it be a good material to make a crystal ball? I tried it, and it seemed to work all right, because after I got the ball made I looked in it and saw some rather startling things.

So, as usual, I am going to stick my neck out and tell you some of the things that I see in this concrete crystal ball.

I predict that at least within the next fifty years Mr. Webster and Mr. ASTM are going to change the definition of an aggregate.

In the first place, Stanton Walker and I will no longer be here to argue about it and hold up progress, and, in the second place, we have known now for some twenty-odd years, I suppose, that concrete aggregate is anything but inactive. As a matter of fact, aggregates are probably the most active pieces of stuff that you can imagine. They shiver and shake, don't they, under the cold winter blasts, and they will squirm

*Presented at the ACI 45th annual convention, New York, New York, February 24, 1949. Title No. 45-32 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE V. 20, No. 8, Apr. 1949, *Proceedings* V. 45. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Aug. 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

†Member, American Concrete Institute; Chief, Research and Geology Division, U. S. Bureau of Reclamation, Denver, Colo.

and turn under the hot summer sun and wind. They will literally jump out of their sockets when they come in contact with a chemical they don't like, even as you and I.

So I think it is about time that Mr. Webster and Mr. ASTM bring their definition up to date.

I see in this crystal ball that our friends, the cement chemists, are no longer in existence. They have become an extinct species and have faded out of the picture. That is because portland cement, as we now know it, has also faded out of the picture. Their places have been taken by prescriptionists who compound and proportion their cementing materials just as carefully and meticulously as do our pharmacist friends of today. Their shelf of chemicals from which they concoct their cementing materials has grown considerably, and, in addition to the common materials you now find there, namely, limestone, shales, clays and iron ores, I see numerous organic compounds, wetting agents, dispersing agents, alkali salts and a whole list of pozzolanic materials. Instead of trying to get alkalis out of cement, as we are concerned with doing now, I see them, or equally impossible substances, being purposely added to the future cements which are concocted of so many materials, and that is because by that time engineers will have learned how to take advantage of the reactivity of aggregates.

When that time comes, I see in the crystall ball that that product, carefully compounded of numerous substances, has, wrapped up in one package, all of the qualities, all of the characteristics that we now find and desire in twenty-some-odd types of portland cement.

I see the answer to a question which an engineer recently submitted to the Institute. This question read as follows: "When shall we take concrete from the specialist, that is, the microscopist, ambitious students, chemists, and chiropractors and give it back to the engineer?" I see the answer very clearly to that question, and, figuratively, the answer is "Never." The reason it is "Never" is because that breed of engineer, the pick-and-shovel transit-and-level type, has also become an extinct species of animal. However, in that same picture, I see that the future world has in reality become an engineer's world, and the pick-and-shovel breed of engineer is now replaced by engineers who have learned to use their electronic, technical tools of engineering to advantage, and they have learned how to use to more advantage the help, advice and assistance that they can get from their friends, the petrographers, the physicists, the atom busters, geologists, and even the biologists.

I see that chemically reactive aggregates are no longer snubbed in elite society. Instead, their presence is cultivated in the best concrete

circles, because our future engineers have learned how to turn reactive qualities to advantageous ends.

I see a complete change in our whole conception of concrete aggregates. Instead of looking at concrete aggregates as pieces of rock and evaluating them as rocks, we are beginning to evaluate them in terms of their actual concrete-making properties. We are not so much concerned with those physical tests that we now use to evaluate our aggregates, but we are more concerned with such things as really affect the concrete-making properties, such things as the type, size and spacing of the pores in the rock, the mineralogical structure, the surface texture, and so on.

I see designing engineers treating concrete as the material it actually is instead of something that they know it isn't. Factors of ignorance have actually changed to factors of safety. I see design methods with both rhyme and reason being followed. Certainly, that isn't the case today. Instead of working stresses of a few hundred pounds per square inch, our designing engineer descendants will be using working stresses in concrete of maybe 40,000 to 50,000 psi—and I didn't get too many zeros on those figures. That will depend on the particular combination of loading conditions with which they are dealing.

I see our present methods, equipment and procedures for manufacturing cement and aggregates and for concrete construction have become as obsolete as the horse and buggy has today. I also see that some of the freakish ideas, as we call them today, of some of our coworkers will have taken their rightful place under the sun.

And persisting throughout this picture, I see in the crystall ball a beam of light. Looking closer, I find that beam of light is the American Concrete Institute. Strange as it may seem, our engineers in the future are going to persist in their interest in this drab stuff that we deal with, and their willingness, as a result of that interest, to spend a lot of time and effort is going to continue to make the American Concrete Institute more and more useful to the engineering profession.

Of course, I can see some pitfalls along the way, some of them, perhaps, in the not too far distant future, but, again, the individual willingness of its members to contribute to the continued growth and success and usefulness of this organization will pull it through.

Now, Mr. President, may I congratulate you on the opportunity that is yours. I am sure you will make a lot more of those opportunities than did your predecessor. But, looking again into this crystal ball, I see another thing or two. The efforts of we pioneer engineers in concrete have long since faded and returned to dust, and in its place I see concrete which actually is worthy of being called permanent. And now, in order

that there may be at least a museum piece for your engineer descendants to look upon with perhaps amusement in the future ages, I charge you with the responsibility of finding some way to preserve this specimen for posterity.

And now, to use some of your own language, Herb, remorselessly, unsympathetically, and with malicious intent, I take pleasure in trading places with you behind this EIGHTBALL.

With halting beginnings in 1916 ready-mixed concrete has grown to an industry which utilized more than 15 percent of the portland cement sold in the U.S. in 1948. Observance of standards and good practice plus cooperation of producer and consumer result in good economical concrete.

Control of Quality of Ready-Mixed Concrete*

By STANTON WALKER†

SYNOPSIS

Control of quality of ready-mixed concrete is discussed. The objective of the paper is to pose the more important problems of concrete control, especially those peculiar to ready-mixed concrete, with the view of eliciting discussion, rather than to attempt to outline recommended practices. Both central-mixing and transit-mixing are discussed. Attention is directed to the mutuality of responsibility of the producers and users in bringing about effective control.

INTRODUCTION

Controlling the quality of ready-mixed concrete presents only a few problems not found in any concrete mixing operation. The use of materials of satisfactory quality and uniformity is, of course, fundamental in any case. The accurate measurement of materials, including water, adequate mixing and suitable handling to minimize segregation are important whether the concrete is job-mixed, centrally-mixed and transported or transit-mixed. However, facilities and procedures for controlling these factors may vary depending upon the type of operation. Certain special conditions peculiar to the ready-mixed concrete operation are of interest.

Ready-mixed concrete falls into two principal classifications—centrally-mixed and transit-mixed. In the first, the concrete is mixed in a central plant and transported to the job in a truck mixer or agitator or other transportation device. In the second, the concrete is mixed in a truck mixer—generally in transit to the job but frequently after the job is reached. Each of these methods of operation has its advantages.

Some very brief history will assist in establishing perspective. The first commercial venture of which the writer has any record was started

*Presented at the ACI 45th annual convention, New York, N. Y., Feb. 24, 1949. Title No. 45-33 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 8, April 1949, *Proceedings* V. 45. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Aug. 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

†Member American Concrete Institute, Director of Engineering, National Ready Mixed Concrete Association and National Sand and Gravel Association, Washington, D. C.

in Baltimore in 1916. Apparently, it did not long survive. In the same year Stephen Stepanian, a member of ACI for many years, filed a petition for a patent on a truck mixer. Two commercial operations were started about 1920 which still survive. One of these was by the West Jersey Sand Co., later taken over by the Warner Co. of Philadelphia. Alexander Foster, Jr., a long-time member of ACI, was president of the first and is vice-president of the second. The other was started by the Fort Worth Sand and Gravel Co. of which R. M. Quigley was president. It continues with the same company name under the presidency of T. E. Popplewell, who, incidentally, is the current president of the National Sand and Gravel Assn. These early operations employed central-mixing. Transit-mixing, while thought of earlier, as evidenced by the Stepanian patent application, did not become established until about 1925. The National Ready Mixed Concrete Assn. was established in 1930. And in 1948 the established industry produced about 25,000,000 cu yd of concrete, utilizing something more than 15 percent of the portland cement sold that year in the United States.

So, we are not dealing with a new and untried industry.

STANDARDS OF PRACTICE

The control of ready-mixed concrete has received attention for many years from ACI, the American Society for Testing Materials, the Portland Cement Assn., the National Ready Mixed Concrete Assn. and others. All have contributed something toward the solution of the problem involved. The first formal specifications for ready-mixed concrete are believed to be those proposed by ACI Committee 504, under the Chairmanship of Miles N. Clair.* Later, by mutual agreement, it was decided that such work was within the scope of the A.S.T.M. Without taking the time to develop the history of the several contributions, attention is called to five current publications directly pertinent to the subject under discussion. They are:

1. *A.S.T.M. Standard Specifications for Ready Mixed Concrete (C 94-48).* The current specifications represent the tenth revision since 1933.

2. *Standards for Operation of Truck Mixers and Agitators,* National Ready Mixed Concrete Assn., approved in 1941.

3. *Truck Mixer and Agitator Standards,* Truck Mixer Manufacturers Bureau, approved in 1944.

4. *Control of Quality of Ready Mixed Concrete,* a 75-page publication of the National Ready Mixed Concrete Assn. outlining practices for concrete control.

*Report of ACI Committee 504, Miles N. Clair, Author-Chairman. "Proposed Specifications for Ready-Mixed Concretes," ACI JOURNAL, Feb. 1930, *Proc.* V. 26, p. 467.

5. *Recommended Practices for Sampling and Testing Ready Mixed Concrete*, a very recent publication of the National Ready Mixed Concrete Assn.

Of course, in addition, there is a large general literature, much of it in ACI *Proceedings*.

No attempt is made here to present a systematic summary of standards of practice. For such detailed information reference to the publications listed is recommended.

PROBLEMS OF CONTROL

If this paper was a well-balanced discussion of the subject given in the title—control of the quality of ready-mixed concrete—the point of the assignment would be missed. It would become bogged down in treating of the problems applicable to any concrete. Therefore, principal attention is directed to factors which appear to be emphasized in the ready-mixed concrete operation or to present special problems of handling.

Most problems of control of ready-mixed concrete revolve about the measurement of mixing water and the companion problem of controlling consistency, which latter is also tied up with length of haul and, or, time of mixing and a number of other things including temperature, rate of mixing, cement, etc. Then there is the question of segregation during discharge and of handling after discharge. Of course, questions are sometimes raised about poorly designed concrete (harsh, under-sanded and the like) but these are matters of completely general application and deserve no special discussion in relation to ready-mixed concrete unless it is to point out that the opportunities for avoiding such difficulties are generally much better in the ready-mixed concrete plant than in the usual job plant.

Air entrainment has introduced some special, but handleable, problems which will be discussed separately from the general picture.

CENTRAL-MIXED CONCRETE

The central-mixing plant has characteristics which more nearly approach those of the job—particularly the large job—plant than the transit-mixing operation. The mixing is done in the same type of mixer as used at the large job. Facilities for batching materials are available which can be matched only in the larger job installations. Opportunities for compensating for moisture content of aggregate and for controlling the consistency of the concrete at the plant are at least as good, and generally better, than in the job operation. A principal difference, which may be of some significance, is that the commercial plant must be flexible

—it must be in a position to batch any one of a large variety of mixes at any time. The job operation, on the other hand, is restricted to only a few classes of concrete and, generally, concrete of a given class will be run in a large number of consecutive batches.

In spite of opportunities for controlling batching and mixing, a special consistency-control problem does exist for the central-mixing operation, especially for long hauls or for conditions where the haul is variable. Mixed concrete stiffens with time and significant reductions in slump may occur. One investigator* states the reduction to average about 1½ in. per hour for concrete hauled in a slowly revolving agitator; he would compensate for that reduction by the addition of about 1 gal of water per cu yd—a perfectly valid procedure. Of course, temperature affects the reduction and another investigator† found, for concrete agitated at about 4 rpm, a reduction in slump of about 1½ in. per hundred "degree hours," where "degree hours" is the concrete temperature in degrees F multiplied by the time in hours. Other factors affect the result—the rpm of the truck mixer or agitator and the grindability of the aggregate being two examples.

For truck mixing or agitating equipment, the A.S.T.M. Specifications limit time of haul to 1½ hours (time between contact of mixing water with cement and completion of discharge) "unless a longer time is specifically authorized"; the specifications recognize that special conditions, such as hot weather, may make it desirable to specify a shorter time.

Adjustment of slump

Without regard to reduction in slump due to time or inaccuracies in control at the central-mixing plant, the desire to be able to adjust consistency at the job always seems to be present. There are a number of reasons for this desire; a very common one is that consumer head-office-policy with respect to limitations on slump differs from those of the job foreman. The specification calls for, say, a 3 to 4 in. slump, the concrete arrives at that slump and the foreman says, "Let's have some water" and there are too few drivers who will withstand the pressure put on them.

Some operators, and some consumers, argue for the desirability of carrying some "tempering water" to permit of such adjustment. The writer does not consider these arguments valid. To carry water for such a purpose defeats a major advantage of the central-mixing operation—centralization of responsibility for controlling the concrete.

*Foster, Alexander, Jr., Knopel, Herbert J. and Whitten, Herbert J., "Ready-Mixed Concrete Operations in Philadelphia," ACI JOURNAL, Feb. 1940, *Proc.* V. 36, p. 363.

†Cook, Glenn C., "Effect of Time of Haul on Strength and Consistency of Ready Mixed Concrete," ACI JOURNAL, April, 1943; *Proc.* V. 39, p. 413.

This problem of adjustment of slump is one for which the consumer must accept joint responsibility and one he can be of outstanding assistance in solving. About as far as the ready-mixed concrete operator has been able to go is to require the representative of the consumer to sign for the extra water—but that is not a very satisfactory solution. The only solution the writer has been able to offer, in the considerable number of years he has been concerned with this subject, is iron-clad adherence to the rule that no water be carried on the agitator and that none be added from any source. Whenever the addition of mixing water is resorted to, control pretty largely goes out the window.

If special circumstances should require that water be added to adjust the slump, care should be exercised to see that it is thoroughly incorporated by about 20 to 30, or more, revolutions of the drum at mixing speed. Without condoning retempering as a practice it should be pointed out that reliable tests have shown that concrete mixed over-long often can be retempered and restored to its original slump with a loss in strength less than indicated by the added water.*

Wash water is frequently carried on an agitator. Whether or not it should be permitted is debatable. There is no theoretical reason against it. Practical objections are that undetected leaky valves may permit water to enter the batch and the temptation is always present to permit the use of "just a little" to adjust consistency. However, if properly handled, there is no valid reason for objecting to it. Further, whether added from the tank on the agitator or from elsewhere, wash water is needed if the agitator is to be kept clean and care should be exercised to see that it is completely discharged before the next batch is loaded. Washing after each load generally is not necessary.

Dump truck transportation

The subject of central-mixing should not be passed without mention of transportation of concrete in non-agitating equipment—the approximate equivalent of ordinary dump trucks. The wide-spread use of air entrainment in concrete has given renewed impetus to that practice and it has been provided for in the A.S.T.M. Standards covering ready-mixed concrete. There has been a lot of clashing of cymbals and gnashing of teeth concerning the matter, but it is doubtful if, really, anything of a revolutionary nature has occurred. Concrete could always be hauled in non-agitating equipment successfully, economically and without damage to the concrete—within certain limitations of conditions. The concrete had to be proportioned about right, the haul couldn't be too long and so on.

*Cook, Glenn C., "Effect of Time of Haul on Strength and Consistency of Ready Mixed Concrete," ACI JOURNAL, April 1943, *Proc.* V. 39, p. 413.

Limitations on the procedure caused it to be largely abandoned except in a few cases where circumstances were favorable. As just stated, the introduction of air entrainment, with accompanying improvement in workability and reduction in segregation, resulted in renewed interest in the question. Also, undoubtedly, the lack of availability of conventional equipment helped to encourage that interest. Now, it seems there is before us the precise problem as before—except that the limitations on conditions are not nearly so severe. There is a place for non-agitating equipment as there always has been. The "size" of that place depends on conditions and will have to be, it seems likely, worked out pretty much for each locality.

TRANSIT-MIXED CONCRETE

There are more transit-mixing operations than there are central-mixing ones. Except as to the stationary mixer, everything that was said about the central-mixing plant is true of the central-batching plant. The facilities for controlling the quantity of ingredients can be as good as for any operation. Control of consistency is again the principal problem which is not present equally in any type of operation.

Similar to the problem of transporting central-mixed concrete, consistency is affected by mixing and agitating rate and time. It is principally affected, of course, by quantity of mixing water and here, in the transit-mixing operation, some quite different, and admittedly difficult, problems are introduced. A major factor is that the control is less centralized.

Consistency as affected by mixing, batching and condition of equipment

Lack of control of rate and time of mixing and agitation can contribute a great deal to lack of uniformity of consistency. If one batch is mixed 10 min at 10 rpm and another 10 min at 5 rpm, the two consistencies are likely to be different. Also, if one batch is mixed 40 revolutions and another 200 revolutions, the consistencies will, again, be different. The number of revolutions of mixing, particularly at the higher speeds, affects the rate of stiffening of the concrete due to the heat which is generated and the grinding of the cement and the aggregate. To insure uniform consistency, the number of revolutions at mixing speed and at agitating speed should be regulated so that they are reasonably uniform from batch to batch. Meticulous control is not necessary but that the length and speed of mixing are factors should be recognized.

Batching procedures affect mixing. The order in which materials are charged into the batch has a bearing on the amount of mixing required, and uniformity in that regard should be maintained. It is desirable, but not always practical, that the arrangement of the batching equip-

ment be such as to permit so-called ribbon loading—that is, the simultaneous loading of the cement and aggregate. Such a procedure is more rapid and at the same time it provides an initial intermingling of the materials which expedites the mixing operation. For that method of loading, the flow of the several materials should be so regulated that they require about the same time for discharge to insure the most efficient intermingling.

The condition of equipment is also a factor. The mixer should be kept clean and free of accumulated concrete mortar and, of course, such admonitions apply to job-mixing and central-mixing as well as to the transit-mixing operation. The condition of the mixing blades should be watched and blades should be replaced when they are worn. The A.S.T.M. Standards suggest a procedure for determining if truck mixers are operating satisfactorily.

Measurement of mixing water

Modern truck mixers, when equipped with water tanks, have dual tanks which are independent of each other—one for the mixing water and the second for the wash water. Both tanks are equipped, or should be, with gages. The mixing water tank is also equipped, or should be, with a device which limits the amount of water which can be discharged without additional manipulation.

It is desirable, but frequently not practical, that mixing water be controlled completely at the proportioning plant. It may be measured there successfully only when the amount of moisture present in the aggregate is known, and taken into account, with accuracy. Under this procedure the required amount of water, and no more, is introduced into the truck mixer tank or directly into the mixer drum.

When knowledge of moisture content is only fairly accurate, and not sufficiently so to permit of complete control of mixing water at the plant, one of two procedures may be followed, with good assurance of satisfactory results. The mixing water tank may be charged with an amount of water which, together with the maximum estimated moisture in the aggregate (and remember, here it is assumed that the moisture content is known with a fair degree of accuracy) will not permit exceeding the maximum quantity of water permitted by the specifications. Then, just enough mixing water is added to produce the desired consistency, and, if conditions have been estimated correctly, this will be somewhat less than is in the tank and than permitted by water-ratio limitations.

A variation of this procedure is to add to the drum somewhat less than known to be required to produce the consistency desired and to carry supplementary water on the truck in a special tank for that purpose—with, of course, adequate provision for accurate measurement of the

supplementary water. The total of the water added to the drum and the supplementary water should be such that the maximum allowable limits cannot be exceeded. When the supplementary water is added to adjust the consistency, the drum should be revolved not less than 20 to 30 revolutions at mixing speed after all of it is in the batch. A common bad practice, when supplementary water is added, is to assume that 3 or 4 revolutions of the drum will thoroughly incorporate it.

When complete dependence is placed on the truck-mixer tank for the measurement of the mixing water, it should be equipped with a gage of satisfactory accuracy and, also, it should be equipped with the device referred to previously which prevents the discharge, without additional manipulation, of more than a fixed amount of water. When wash water is carried on the truck it should always, as previously stated, be in a separate tank designed for that purpose. Except as outlined below, water from that tank should not be used as a part of the mixing water.

When the wash water is permitted to be retained in the drum for reuse as a part of the mixing water for the succeeding batch, it should be done only under the most careful supervision and, under these circumstances, control should be set up as a central plant function. There is no theoretical objection to using the wash water as a part of the mixing water for the next batch if it is free from foreign matter such as hardened and partially set lumps of concrete broken free from the mixing drum in the cleaning process and if it is accurately measured. The latter, of course, is the rub. Nevertheless, it is sometimes difficult to find a place to discharge it and, further, it should not be over-looked that its discharge discards some cement and other fine material adhering to the truck mixer drum.

Leaky water valves may contribute to variable consistency although this is less common than before the advent of water pumps for introducing the mixing water. Water valves can be maintained so they will not leak but they do require attention. There is no doubt that many of the instances of the latter part of the batch appearing wetter than the first part, especially when discharge is slow and intermittent, have come from water entering the batch during discharge.

CONTROL OF WATER BY CONSISTENCY

Further comment on water control as applied to any operation is of interest. The only significant obstacle is, of course, lack of knowledge of free moisture available in the aggregates. There are a number of procedures, which are reasonably accurate and rapid, for the determination of moisture content. They include simple drying, measurement of displacement, various electrical devices and others. However, all of them

make use of relatively small samples and none of them is rapid enough for batch to batch determinations. Accordingly, the end result is no more accurate than the sample is representative of the materials used between determinations. That suggests attention to handling aggregates so that the moisture content is uniform, or at least, does not change rapidly. It also suggests the need for better procedures for determining moisture.

With other factors controlled in a uniform fashion, consistency alone is an excellent measure of mixing water. That is to say, having estimated the quantity of water required to produce a given slump, with other conditions the same, successive batches of the same slump will contain substantially the same amount of water. This suggests a greatly needed development in equipment—*i.e.*, a reliable consistency meter. A number of not-too-satisfactory ones have been tried out and others are in the experimental stage. In the meantime the fact that the slump test, expertly made, represents a quite dependable measure of consistency should be kept in mind.

It is interesting to consider just what effect variations in moisture content in the aggregates have on the concrete if the consistency is accurately controlled. Calculations show that the cement factor and water ratio for average concrete can be maintained within the limits of about ± 2 percent for ranges in moisture content of as much as 2 to 8 percent in the fine aggregates and 0 to 4 percent in the coarse aggregates.

DISCHARGE CONTROL

Discharge control is of outstanding importance for both the agitator and the truck mixer. All of the advantages to be gained from meticulous control of operations prior to discharge may be thrown away by careless or improper discharge methods. The mixer or agitator should be rotated in the discharge direction and the rate of discharge controlled by the speed of the drum. It should never be controlled by varying the discharge opening since that tends to induce segregation.

Unrestricted discharge at full drum speed, with a full discharge opening, is desirable whenever the concrete can be disposed of at the maximum discharge rate—about 15 to 20 seconds per cu yd. When discharge is intermittent, as into wheelbarrows, buggies, etc., the rate should be controlled, as already said, by manipulation of drum speed. Under these circumstances, it sometimes may be desirable to require a few revolutions in the mixing direction in the interval between discharging operations.

Flat chutes should be avoided unless the concrete is helped along by means of shovels or hoes. Long chutes should also be avoided. In discharging from chutes or spouts, the discharge end should be near the

surface of the concrete and should be swung back and forth to avoid coning and consequent segregation. In a road slab, a spreader is most helpful in distributing the concrete with a minimum of segregation.

AIR ENTRAINMENT

The advent of air entrainment has introduced new control problems, at least one of which is peculiar to the ready-mixed concrete operation. It is, of course, important to maintain the amount of air entrained in any concrete mixing operation within fixed limits. If air content is too high, the strength of the concrete is unjustifiably, and in some cases dangerously, reduced. If it is too low the potential improvements in durability and other characteristics are not attained.

It would be entirely appropriate to discuss such general questions as inter-ground additives *vs.* admixtures, effect of proportions and aggregate grading, methods of measuring air content, etc., but it would be too space-consuming. Therefore, only brief mention will be made of the principal factor which is different in the ready-mixed concrete operation —the time element. The amount of air is affected by time of mixing or agitation. The maximum amount of air is entrained within a relatively short time and thereafter it is reduced by the mixing action. Circumstances affect the results too much and quantitative data are too few to permit detailed discussion. However, both laboratory and field tests suggest the factor of time of mixing to be sufficiently important to require careful attention. Limited laboratory data suggest that some admixtures may produce air bubbles which are more tenaciously held by the concrete than others.

CLOSURE

The mutuality of responsibility of the producer and consumer of ready-mixed concrete can not be over-emphasized. That should be evident from a review of what has been said. The consumer must assist the producer by scheduling his work so that reasonable uniformity in the time element is maintained. He should cooperate to make conditions for discharge favorable and he must accept full responsibility for the concrete after its discharge. The function of the ready-mixed concrete producer is to furnish freshly mixed concrete to the purchaser at the job site. In his regular capacity he has no control over what may happen to the concrete after it is discharged from the transportation vehicle.

This paper summarizes Bureau of Reclamation test results and recommendations concerning lightweight aggregates produced in western United States. A report of the Bureau of Standards investigation of eastern aggregates will appear in the May issue.

Tests of Lightweight-Aggregate Concrete Designed for Monolithic Construction*

By WALTER H. PRICE and WILLIAM A. CORDON†

SYNOPSIS

The characteristics and origin of lightweight aggregates are described and the comparative results of laboratory tests of strength, insulating value, shrinkage, and weathering resistance of concrete made with lightweight aggregates from 17 producers are summarized. Data are included on expanded shale, expanded slag, scoria, pumice, perlite, exfoliated vermiculite and diatomaceous earth.

INTRODUCTION

Lightweight concrete is an important building material which has been successfully employed in construction in this country for more than 50 years. It has many advantages not possessed by regular sand-gravel concrete. It is lighter in weight, ranging from 35 to 115 lb per cu ft; and savings in structural steel and foundation size result from its use. Lightweight concrete has better fire resistance properties, is a better insulator against sound and heat, and is as weather resistant as regular concrete. Its strength, which is roughly proportional to its weight, ranges from 200 psi for the lighter concretes to over 5000 psi for the heavier ones. Lightweight concretes usually cost more than regular concrete because of the higher cost of the aggregates, greater cement requirements, and the need for greater care in placing. Only those lightweight concretes made from lightweight aggregates will be considered.

The use of lightweight concrete in building construction has been stimulated in recent years by the scarcity of other building materials, the saving in over-all construction costs which sometimes results from its light weight, and the experience which has been gained in its use. A typical example is the use of Gravelite lightweight-aggregate concrete

*Received by the Institute Mar. 10, 1949. Title 45-34 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 8, Apr. 1949, *Proceedings* V. 45. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Aug. 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

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in the upper deck of the San Francisco-Oakland Bay Bridge. Lightweight concrete is used for floors and as fire protection in office buildings and apartment houses and in the construction of dwellings.

The lack of comparable test results on many of the lightweight aggregates considered for housing construction prompted the Housing and Home Finance Agency to sponsor and finance a laboratory research program to investigate the properties of lightweight concrete covering the range of aggregates now on the market, with particular attention to some of the newer products. The tests were conducted in the laboratories of the National Bureau of Standards and the Bureau of Reclamation. Nineteen samples of lightweight aggregate from seventeen producers in western United States, including seven general types of aggregates, were tested in the laboratories of the Bureau of Reclamation. A brief summary of these tests* is included in this paper.

TYPES OF LIGHTWEIGHT AGGREGATE

Lightweight aggregates are produced by expanding clay, shale, slate, diatomaceous shale, perlite, obsidian, and vermiculite through the application of heat; by expanding blast-furnace slag through special cooling processes; from natural deposits of pumice, scoria, volcanic cinders, tuff, and diatomite; and from industrial cinders. Such aggregates, sold under many trade names; are identified and described briefly.

Cinders

Cinders are the residue from high temperature combustion of coal or coke in industrial furnaces. The residue from domestic furnaces is not considered suitable for concrete aggregate. The Underwriters Laboratories limit the average combustible content of mixed fine and coarse cinders for manufacturing precast blocks to not more than 35 percent by weight of the dry mixed aggregates. They also require that the sulfides in the cinders be below 0.45 percent and the sulfate be less than 1.00 percent. Stockpiling of cinders to permit washing away of undesirable sulfur compounds is recommended. However, exposure to weather will in no way reduce the unsoundness due to the presence of unburned coal. Cinders have been used, wherever available, in concrete construction with satisfactory results for more than 50 years. All-cinder concrete weighs about 85 lb per cu ft and when sand is used to increase the workability in monolithic construction it weighs from 110 to 115 lb per cu ft. A 1:2:5 cinder concrete having a unit weight of about 108 lb per cu ft and a minimum 28-day strength of 800 psi was specified for the monolithic concrete placed in the floors of the Stuyvesant Town, New York, apartment houses.

*For detailed test results see *Materials Laboratories Report* No. C-385, which was furnished to the Housing and Home Finance Agency.

Expanded slag

Expanded slags are produced by a number of methods. In one method the molten slag is run into pits containing a predetermined amount of water, which becomes steam and vesiculates the slag. In another method, the stream of molten slag is broken up by a revolving table, from whence it encounters water sprays and is expanded as it flows through an open trough and conveyor to the stockpile. In a third method, the molten slag flows from the blast furnace into the top of a machine, in which it is broken up by a number of paddles and blades revolving horizontally. This violent agitation, plus the steam from a manually controlled stream of water sprayed on the blades, paddles, and plates of the machine, expands the slag. The amount of water used in these processes has a very pronounced influence on the resulting product. Some of the names under which expanded slag has been or is being sold are: Calicel, Celocrete, Foamed Slag, Pottsco, Slaglite, Superrock, Tuff-Lite and Waylite. Concrete made with aggregate consisting entirely of expanded slag weighs from 75 to 100 lb per cu ft. The strength of expanded slag concrete varies from 200 to 2500 psi depending on the slag and cement content of the mix.

Expanded shale and clay

Expanded shale and clay aggregates are made by heating certain prepared clays and shales to the fusion point where they become soft and expand because of entrapped expanding gases. With the exception of Haydite, the raw material is crushed or preformed to the desired size before heating takes place. Better known of such aggregates are:

Haydite—In making Haydite the raw shale is crushed to pass through a screen having 1½-in. openings. After expansion it is crushed to the desired sizes. Very good structural concrete can be made with Haydite aggregate. It was used in the construction of ships during the war; when mixed with a natural sand strengths ranging up to 9000 psi at 1 year age for unit weights of about 112 lb per cu ft were obtained. Concrete made from Haydite has a unit weight ranging from 100 to 110 lb and ordinarily possesses strength ranging from 1000 to 5000 psi, depending on the richness of the mix.

Rocklite—In the manufacture of Rocklite the shale is crushed to the desired size before heating. The resulting product has a tight skin and high strength, and a very good structural concrete can be produced with it having unit weights varying from 90 to 100 lb per cu ft and strengths ranging from 1000 to 5000 psi.

Gravelite—Prepared clay or shale is extruded through a die to form cylinders which are cut to desired length. The small cylinders of clay which are sprayed with fuel oil become spherical in shape and expand

when heated in a rotary kiln. Pellets below ¼ in. in size are formed by controlling the water so that pellets of the desired size are formed during the mixing process. All-Gravelite concrete weighs from 75 to 85 lb per cu ft. Where most of the fine aggregate is natural sand, the weights vary from 95 to 110 lb per cu ft. Strength of Gravelite concrete used in the San Francisco Bay Bridge averaged about 3800 psi.

Cel-Seal—This is a pellet-shaped expanded clay aggregate. Billets of the desired size are passed through a revolving cylinder in which they are rounded and coated with powdered refactory clay that has a higher fusion temperature than the main body of the pellet. The pellets are then expanded in a rotary kiln at a temperature above the fusion point of the pellet but below that of the refactory coating. Cel-Seal was formerly known under the trade names Globulite and Vesiculite. It has been made experimentally.

Nodulite—Another pellet-shaped aggregate which was made from Fuller's earth in Florida during the war in connection with the concrete ship construction.

Claydite and Featherlite—Other expanded clay aggregates.

Processed diatomaceous shales

Lightweight aggregates known as Diacrete and Raylite are made from diatomaceous earth or shales by crushing and screening to desired sizes, spraying with oil, and burning in a rotary kiln.

Airrox—This aggregate is made from an oil-impregnated shale under two processes. In one process the raw shale is crushed and screened to desired sizes and coated with powdered limestone as it is heated in a rotary kiln. The lime coating combines with the shale to form a thin hard coat of calcium silicate. In the second process, Airrox is produced by heating raw crushed shale in open beds at a temperature below fusion.

Expanded slate

A patent was recently issued covering the production of lightweight aggregate from waste slate piles. In this process the slate is crushed and screened to size and coated with a refactory material to prevent agglomeration during heating in a rotary kiln. The refactory material is separated from the burned slate and reused.

Sintered aggregate

Several patents cover the processes of producing lightweight aggregate by burning mixtures of coal and shale, clays, cinders, fly ash, slag, or ashes in moving grates exposed to flames under forced drafts. The resulting product is crushed to the desired sizes. Four of these aggregates are:

Corlite—A product which has been made experimentally by sintering chemically treated anthracite ashes.

Sinter-lite—A lightweight aggregate made by sintering mixtures of fly ash and slag resulting from the burning of powdered coal in industrial plants.

Lelite—A lightweight aggregate made by sintering a mixture of anthracite and shale.

Lytag—A lightweight aggregate which was produced experimentally by sintering a mixture of crushed clays, water, and granulated coal.

Natural aggregates

Pumice, scoria, volcanic cinders, tuff, and diatomite are the only common natural rocks that are light and strong enough to be used as lightweight aggregate without processing other than crushing and screening to size. Diatomite is the only one not volcanic.

Pumice—The most widely used of the natural lightweight aggregates. It is a highly vesicular froth-like volcanic glass which is usually white-gray to yellow in color but may be red, brown, or even black. It is found in large beds in western United States and is produced as a lightweight aggregate in California, Oregon, and New Mexico. Concrete made with sound pumic aggregate weighs from 65 to 90 lb per cu ft, and the strength ranges from 500 to 3400 psi. The quality of structurally weak pumice having high absorption characteristics may be improved by calcining at temperatures near the point of fusion. Some of the names under which pumice is sold are Basalite, Pyramid, Ingham, Insulpum Agite, and Vocolite.

Scoria—A highly vesicular lava in which the vesicles are rounded or elliptical in cross section as contrasted to pumice, in which the vesicles are tube shaped and approximately parallel. It is at present being produced as a lightweight aggregate from deposits in New Mexico. One scoria aggregate is sold under the trade name of Volscor. Scoria aggregate resembles industrial cinders and is usually red to black in color. Concrete made with scoria aggregate weighs from 75 to 95 lb per cu ft, with strengths ranging from 500 to 3000 psi.

Diatomite—A consolidated sedimentary rock composed primarily of diatom skeletons (diatomaceous earth).

Tuff—A consolidated volcanic ash.

Volcanic cinder—A loose agglomerate composed largely of highly vesicular (scoriacious), glassy fragments of lava, 4 to 32 millimeters in size; also the individual fragments of this description as they occur in ash, tuff, agglomerate, or breccia.

Expanded obsidian, perlite, and vermiculite

When obsidian is heated to the temperature of fusion, gases are released which expand the material. The interior of the particles produced is of vesicular structure and the surfaces are smooth and quite impervious. The raw material is crushed and screened to size and coated with a fine material of higher fusion characteristics to prevent agglomeration during the heating process. Expanded obsidian has been produced experimentally.

Perlite—The rock from which perlite lightweight aggregate is manufactured has a structure resembling tiny pearls compacted and bound together. When perlite is heated quickly it expands with disruptive force and breaks into small expanded particles. Usually, expanded perlite is produced only in the sand sizes. Concrete made with expanded perlite has a unit weight ranging from 30 to 65 lb per cu ft and strengths ranging from 100 to 1600 psi. It is a very good insulating material. Some of the trade names under which expanded perlite is sold are: Permalite, Superlite, Ryolex, CBM Aggregate, and PerAlex.

Exfoliated vermiculite—Vermiculite is an alteration product of biotite and other micas and is found in Colorado, Montana, and North and South Carolina. The color is yellowish to brown. Upon calcination, vermiculite expands at right angles to the cleavage into a fluffy mass, the volume increasing as much as 30 times. It is a very good insulating material and is used extensively for that purpose. It is also sold under the trade names of Alexite, Vosicon, Vesco, and Zonolite. Concrete made with expanded vermiculite aggregate weighs from 25 to 55 pounds per cu ft, and has strengths ranging from 50 to 700 psi.

LABORATORY TESTS

Scope

Seven general types of lightweight aggregate were included in the nineteen samples tested in the laboratory of the Bureau of Reclamation. The nineteen samples were divided as follows: (1) two samples of expanded shale, one expanded before crushing, the other expanded and coated after crushing; (2) one sample of scoria; (3) three samples of expanded slag; (4) five samples of pumice, one a finely divided pumicite, and one which had been treated by calcining; (5) five samples of expanded perlite; (6) two samples of exfoliated vermiculite, one received with air-entraining agent mixed in sample; and (7) one sample of diatomaceous earth (diatomite). The samples were furnished by seventeen producers.

The aggregates were tested for specific gravity, absorption, grading, petrographic structure, crushing strength, and rodded unit weight.

Concretes made with the aggregates were tested for compressive and flexural strength, elastic properties, resistance to alternate cycles of

freezing and thawing, resistance to alternate cycles of wetting and drying, absorption, drying shrinkage, thermal expansion, thermal conductivity, expansion resulting from alkali-aggregate reaction, and nailing and sawing properties. The fresh concretes were also compared as to workability and improvement in workability resulting from the entrainment of air.

Description of tests

The procedures outlined in ASTM Designation C 127-42 and C 128-42 were followed as far as possible in determining the absorption and bulk specific gravity of the aggregates. The unit weight of each of the lightweight aggregates was determined by filling a measure in two equal layers and rodding each layer only 10 times to minimize breakage. The crushing strength of the aggregate was determined by compacting the aggregate lightly with a 1-in. diameter wooden rod to a depth of 5 in. in a 3 x 6-in. cylinder and then applying a load through a piston fitted in the top of the cylinder. The load required to compress the aggregate 1, 2, and 3 in. was used as a measure of the strength which might be obtained when the aggregate is incorporated in concrete. All aggregates were examined by petrographic methods to determine whether they were potentially reactive with the alkalies contained in portland cement.

Preliminary trial mixes were made with each lightweight aggregate to determine the water requirement necessary to produce plastic mixes of approximately 3-in. slump for each of 3, 5, 7 and 9 sacks of cement per cu yd. The aggregates were batched in a room-dry state and adjustments were made for absorption. Mixes were tried with the aggregates as received, and this grading was used in the concrete mixes which followed if a workable concrete could be produced. If a workable mix could not be produced, the grading was altered as little as possible to produce a workable concrete. The gradings of the aggregates as used in the concretes are shown in Table 1. No attempt was made to alter the grading of the fine perlites, vermiculites, or pumicite. Each aggregate was also tested in a concrete mix containing an air-entraining agent. All tests were made at 70 F unless otherwise noted.

Compressive and flexural strengths and modulus of elasticity of the concrete were determined on 6 x 12-in. cylinders and 4 x 4 x 20-in. bars cast from 3, 5, 7 and 9-sack mixes and cured under the following conditions: 7-day fog curing, 7-day fog curing followed by 21 days in 50 percent humidity room, 7-day fog followed by 21 days in 50 percent humidity room and 14 days of drying at 122 F and 15 percent relative humidity.

Drying shrinkage determinations were made on 4 x 4 x 40-in. bars cast from the 3, 5, 7 and 9-sack mixes for the following curing condi-

TABLE 1—PHYSICAL PROPERTIES AND GRADINGS OF LIGHTWEIGHT AGGREGATES

Standard sieve size	Expanded shale No. 1	Expanded shale No. 2	Pumice No. 1	Pumice No. 2	Pumice No. 3	Pumice No. 4	Pumicite	Scoria	Slag No. 1	Slag No. 2	Slag No. 3	Perlite No. 1	Perlite No. 2	Perlite No. 3	Perlite No. 4	Perlite No. 5	Vermiculite No. 1	Vermiculite No. 2	Diatomaceous earth
Coarse aggregate grading—percent retained																			
¾—1½-in.	1	0	0	0	5	0		0	0	0	0								0
⅜— ¾-in.	48	50	50	5	55	0		0	50	0	0	4			0		0		40
No. 4—⅜-in.	51	50	50	95	40	100		100	50	100	100	96			100		100		60
Sand grading—percent retained																			
No. 8	13	10	25	31	13	15	0	21	1	19	38	40	5	0	20	0	18	12	9
No. 16	25	20	27	24	18	20	0	18	8	32	34	21	26	8	31	0	34	52	31
No. 30	25	25	18	17	15	19	0	13	18	28	18	12	35	50	11	13	28	10	26
No. 50	10	23	7	9	13	14	0	15	23	13	7	5	26	29	8	30	15	12	20
No. 100	6	13	6	5	11	9	1	13	15	4	2	4	4	9	8	28	4	8	14
Pan	5	9	17	14	30	23	99	20	35	4	1	18	4	4	22	29	1	6	0
Percent sand	50[1]	50	42	88[2]	55	93	100	63	50	96	90	73	100	100	95	100	96	100	27
Physical properties (samples as received)																			
Percent passing No. 200 sieve	3.0	3.0					91			3.6	0.9	5.7		2.9		14.2	2.7	3.6	
Bulk specific gravity	1.47	1.86	1.70	1.47	1.23	1.65	2.34	2.30	1.18	2.22	2.28	0.88	0.88	0.67	0.99	0.88	0.89	0.90	1.29
24-hr absorption—percent	8.5	15.8	8.0	21.24	18.3	31.9	0.7	8.7	8.4	18.0	10.3	11.2	12.1	12.4	30.7	51.	25.7	19.8	50.1
Unit weight lb/cu ft	47.64	51.27	47.65	47.0	30.26	43.89	53.65	65.0	25.78	33.64	40.82	16.12	15.28	9.95	10.18	2.94	12.14	8.05	22.89

[1]16 percent by weight No. 50 and No. 100 natural sand used in non-air-entraining mixes which equals 10 percent by volume.
[2]42 percent by weight natural sand used in non-air-entraining mixes which equals 28 percent by volume.

tions: 7 days fog followed by storage at 50 percent relative humidity for periods up to 6 months, 7 days fog followed by 21 days storage at 50 percent relative humidity, and 14 days at 15 percent relative humidity at 112 F.

Thermal coefficient of expansion determinations were made on 4 x 4 x 40-in. bars sealed in copper jackets. Specimens were cast only from the 7-sack mixes for this test. The specimens were stored at 70 F for 28 days, followed by 10 days at 122 F after which they were alternated between 70 and 122 F each day until 4 cycles were completed.

Wetting-and-drying tests were conducted on 3 x 3 x 15-in. bars which were subjected to daily cycles of soaking for 16 hours in water at a temperature between 70 and 80 F and drying in air at 130 F. Prior to testing, bars were fog cured for 7 days followed by 21 days storage at 50 percent relative humidity and 2 days soaking in water. Wetting-and-drying tests were made on bars from the 5 and 7-sack mixes.

Accelerated freezing-and-thawing tests were made on 3 x 6-in. cylinders cast from the 5 and 7-sack mixes. The specimens were fog cured for 14 days followed by 14 days at 50 percent relative humidity and 2 days of soaking in water before starting the alternate cycles of freezing and thawing. A cycle consisted of 1½ hours of freezing at 10 F followed by 1½ hours of thawing at 70 F, all in water. Eight cycles were obtained each 24 hours.

A guarded hot plate conforming to ASTM Designation C 177-45 was used in making the thermal conductivity determinations. The discs were fog cured for 30 days followed by 60 days storage at 50 percent relative humidity. The specimens were then ground smooth on each face and oven dried to constant weight at 150 F before testing. The specimens were about 270 days old when tested. Duplicate determinations were made at 83 and 130 F.

Pieces of 4 x 4 x 40-in. bars used in the other tests were used to determine the nailing and sawing properties of the lightweight concrete. A hacksaw was used in the sawing tests. The sawability was classed as excellent, very good, good, fair, or poor, depending upon the amount of downward pressure exerted on the saw to cause cutting and the ease and rapidity with which the saw cut into the concrete.

Since nails could be readily driven into some of the lightweight concretes but were not held firmly, the nailing test consisted of two parts, driving and holding. Where possible, three eight-penny nails were driven into the bar to half its depth. Nailing properties were judged by the blows required to drive the nail, the splitting or cracking of the bar during the driving, and the force required to remove the nail.

DISCUSSION OF TEST RESULTS

It is difficult to produce workable concrete with most lightweight aggregates because of the angular and vesicular structure of the particles, the nonuniformity of the grading as furnished, and the tendency of the aggregate (due to its lightness) to separate from the mix. Gradings which have been found suitable for sand-gravel concrete are satisfactory for lightweight concretes; however, lightweight aggregates should be graded on an absolute volume basis because there may be considerable difference in the specific gravity of the fine and coarse particles requiring a proportionately large amount of fines on a weight basis. Workability of lightweight concrete can be improved by crushing the coarser sizes to improve the sand sizes, by adding natural sand, or by using air-entraining agents.

The use of air-entraining agents does not increase the weight of the concrete and is very effective in counteracting large deficiencies in the aggregate grading. The amount of air entrained in lightweight concrete mixes is very difficult to determine since an air meter will measure not only the entrained air but also the unfilled surface voids of the aggregate. In addition, the theoretical unit weight used in the gravimetric method of air determination cannot be determined with any degree of accuracy. The air contents of the lightweight concrete mixes, estimated by comparison, with mixes containing no agent, varied up to about 20 percent by volume, depending upon the aggregate. This amount of air was produced by using approximately twice as much agent as normally used in a 1½-in. sand-gravel mix for 4 percent air. Entrained air in lightweight concrete apparently does not reduce the strength as much as it does in sand-gravel concrete. Without the use of entrained air it would be very difficult to obtain a workable mix with some of the harsher aggregates.

Because of the tendency of lightweight aggregates to separate from the paste, the lowest practicable slump commensurate with placing conditions should be employed at all times. A maximum slump of 2½ in. should be adequate for all air-entraining lightweight concrete.

It has already been mentioned that the aggregates used in these tests were batched in a room-dry state. This dry batching is contrary to the procedure of saturating the aggregate before batching, as recommended by some of the lightweight aggregate associations. The results of these tests indicate that the strengths are not materially affected by the dry aggregate as compared to saturated aggregate. Tests made during the concrete shipbuilding programs at the Oakland, California, yard showed about 7 percent increase in strength in favor of the dry aggregate. Also, it has been shown that concretes made with dry aggregates are much more resistant to freezing and thawing than those made with saturated aggregates. The Oakland tests also showed that the concrete made

with the dry aggregate was about 3 percent lighter and that this concrete retained this difference in weight after a year of soaking.

The strengths, thermal conductivity values, and unit weights shown in Table 2 have been rearranged in Table 3 to show the strength which can be obtained with each aggregate tested, the cement content required to produce this strength, and unit weight and thermal conductivity of the concrete for a given strength. From Table 3 it can be seen that concrete having a 28-day compressive strength of 5000 psi can be produced with only two of the aggregates, and that concrete having a compressive strength of 3000 psi can be produced with only five aggregates. The cement required to produce a strength of 1000 psi varies from 2 to 10 sacks per cu yd of concrete depending on aggregate used. In general, the strength of lightweight concrete can be increased through the addition of natural sand; however, this addition also increases the weight of the concrete. It is not possible to produce strengths much greater than 700 psi with exfoliated vermiculite and the finer graded perlites. These concretes, however, have very good insulating properties, are considerably lighter than the stronger concretes, and are strong enough for many types of construction.

The modulus of elasticity of most lightweight concretes tested was found to be only about one-half as great as those of natural aggregate concrete of the same strengths. The flexural strength of regular concrete is about one-tenth its compressive strength. Many of the weaker lightweight concretes produced flexural strengths as high as one fourth of their compressive strength, and, in most cases, the flexural strength of lightweight concrete is comparatively higher than that of regular concrete of the same compressive strength.

The drying shrinkage of lightweight concrete is, in general, greater than that of regular concrete because more water is used in mixing the lightweight concrete and because the aggregate is softer and offers less resistance to the shrinkage forces. Lightweight concrete, however, should be able to shrink more than regular concrete without cracking because of its lower modulus of elasticity and because of its comparatively higher tensile strength. Some lightweight aggregates which have very high drying shrinkage values can be expected to crack when used in slabs of any length. The concrete made with crushed diatomaceous earth aggregate had a drying shrinkage five times that of regular concrete. Concrete made with this aggregate cracked and showed separation around the aggregates in the beams and specimens used in the tests. One vermiculite concrete also had a drying shrinkage about five times as great as regular concrete. This concrete showed no distress in the specimens nor did any of the concretes made with other lightweight

TABLE 2—LIGHTWEIGHT CONCRETE

Material	Cement content sacks/cu yd	Water content lb/cu yd	Slump in.	Unit wt fresh concrete lb/cu ft	Vinsol resin grams/cu yd	Comp. strength psi: 7-day fog	7-day fog 21-day 50 percent humidity	7-day fog, 21-day 50 percent humidity 14-day 122 F	Flexural strength psi 180-day	E x 10^{-6} 7-day fog 21-day 50 percent humidity 14-day 122 F	Unit wt cured conc, lb/cu ft 42-day
Expanded shale and clay Sample No. 1	3.36	380	2.0	89.1		725	1290	1505	270[6]	1.43	79.6
	5.17	399	2.3	93.3		2090	3265	3435	465	1.67	84.3
	7.10	423	4.7	97.9		3010	4410	4620	570	1.88	90.2
	7.10	368	3.7	92.8	82.3	3165	4345	4695	500	1.63	87.4
	8.52	421	3.5	99.6		3650	4755	5495	515	1.72	94.0
Expanded shale and clay Sample No. 2	3.42	502	2.5	102.1		450	1025	1045	255[6]	0.99	90.5
	5.06	481	3.5	104.9		1155	2060	2510	435	1.67	93.9
	7.11	468	4.3	108.7		2545	4005	4830	615[7]	2.19	103.3
	7.66	500	3.8	102.4	85.7	2410	3880	4105	610	1.86	97.0
	8.91	461	4.0	109.5		3740	4970	5295	685	2.05	104.4
Expanded slag Sample No. 1	3.13	612	2.5	93.1		175	425	530	190[6]	0.57	75.9
	5.10	585	4.5	96.4		565	1130	1220	310	0.93	83.3
	7.18	562	5.0	99.9		1175	1845	2240	475	1.40	91.6
	7.08	425	3.5	79.5	76.7	855	1420	1415	285	0.81	75.3
	8.94	517	3.5	103.6		1655	2485	2390	545	1.42	96.1
Expanded slag Sample No. 2	5.33	222	2.0	74.5	78.5	180	300	260	95	[9]	63.9
	7.43	417	3.5	84.0	72.0	450	775	630	225	[9]	74.7
Expanded slag Sample No. 3	5.14	287	2.0	82.4	80.6	300	580	580	100	[9]	80.2
	6.82	367	4.2	93.0	75.4	390	765	655	190	[9]	82.5
Scoria	2.95	440	4.5	77.7	69.3	75	190	220		[9]	
	5.23	470	4.1	89.7	74.1	735	780	820		[9]	
	7.21	503	2.2	101.4	85.7	1115	1875	1895	460	1.20	93.9
	9.17	509	4.7	102.3	69.5	2140	2540	2980		1.16	
Pumice (treated) Sample No. 1	3.15	681	6.0	91.4		175	515	505		[9]	70.6
	4.90	633	3.5	95.6		850	1700	1695	275	0.83	78.7
	6.93	624	4.2	99.2		1655	2705	2845	330	0.96	84.1
	7.10	551	4.3	94.6	83.5	1520	2740	2570	360	0.91	82.8
	9.27	235	4.2	99.2		2155	3210	3220	325	1.05	85.6
Pumice Sample No. 2	3.11	531	1.4	90.9		285	350	710		[9]	74.0
	5.08	516	1.5	95.2		825	1335	1760	375	1.01	82.0
	7.06	496	2.5	98.0		1965	2520	2865	380	0.85	87.9
	7.28	478	3.7	82.4	80.0	1820	2205	2170	355	[9]	73.1
	8.74	507	4.2	100.6		2565	3175	3265	210	0.97	90.9
Pumice Sample No. 3	5.06	565	3.0	86.8		610	1185	1285	170	0.37	72.9
	7.15	593	4.3	90.3		1130	1805	1605	210	0.36	78.8
	7.00	566	3.3	88.3	77.3	1150	1765	1495	260	0.44	66.4
Pumice Sample No. 4	5.04	632	2.2	92.0		255	585	630	68	[9]	68.1
	7.28	707	2.8	96.2		705	1485	1605	200	0.19	76.8
	6.92	783	3.0	93.8	73.0	750	1490	1590	195	[9]	80.8
Pumicite	5.05	950	4.0	101.7		335	895	780	135	[9]	74.3
Perlite Sample No. 1	5.09	430	1.5	51.9	76.5	365	530	575		[9]	43.5
	6.86	469	2.5	59.0	77.9	665	970	870	160	0.33	51.0
Perlite Sample No. 2	3.16	876	1.0	61.2		35	70	85		[9]	32.6
	5.08	831	1.0	65.3		110	205	230	65[7]	[9]	40.4
	7.36	790	3.5	71.9		250	700	690	240	0.30	51.0
	7.21	579	5.0	61.7	74.6	520	835	815	190	[9]	48.6
	8.98	686	3.5	72.4		965	1490	1325	245	0.23	58.9
Perlite Sample No. 3	4.78	795	2.8	58.6		115	230	110		[9]	33.1
	6.92	739	4.5	63.4		360	640	615	155	0.18	44.2
	7.05	558	4.0	55.5	80.2	475	720	700	100	0.17	39.5
Perlite Sample No. 4	5.79	819	1.3	73.5		175	265	230	190	[9]	51.5
	7.08	823	3.0	76.3		250	435	465	200	0.26	58.3
	7.01	579	1.0	62.8	72.5	300	495	490	140	[9]	51.8
	9.44	758	3.0	81.9		405	610	810	300	0.44	62.4

MIX DATA AND TEST RESULTS

Comp. strength wt ratio psi/lb	Absorption percent by dry wt	Freezing & thawing: Cycles to failure[2]	Wetting & drying: Expansion percent[3]	Alkali expansion 12-mo	Drying shrinkage percent 80-day	Thermal coeff. of expansion x 10^6	Thermal conductivity k[4]	Aggregate crushing strength[5]	Nailability	Sawability	Workability
18.9				Neg	−.0129[6]			890	Good	Good	Fair
40.7	10.4	390	.022	Neg	−.0460			890	Fair	Fair	Good
51.2	9.3	600[8]	.021	Neg	−.0532	4.9	3.47	890	Poor	Poor	Very good
53.7	8.4			Neg	−.0518			890	Very poor	Very poor	Very good
58.5	8.5			Neg	−.0532			890	Very poor	Very poor	Very good
11.6					−.0150[6]			1060	Fair	Fair	Poor
26.7	12.2	190	.025		−.0435		4.21	1060	Poor	Poor	Fair
46.7	21.5	600[8]	.023		−.0552[7]	5.3	4.11	1060	Very poor	Very poor	Good
42.3	7.2				−.0592			1060	Very poor	Very poor	Very good
50.7	12.4				−.0562		3.73	1060	Very poor	Very poor	Very good
7.0				Neg	−.0025[6]			85	Fair	Excellent	Poor
14.6	16.9	140	.013	Neg	−.0602		2.64	85	Good	Very good	Fair
24.5	15.1	430	.012	Neg	−.0760	5.3	2.59	85	Very good	Very good	Good
18.8	13.0			Neg	−.0830			85	Very good	Very good	Very good
24.9	14.7			Neg	−.0738			85	Good	Good	Very good
4.0					−.1628			35	Poor	Very good	Good
8.4	22.9				−.1722			35	Fair	Very good	Good
7.2	20.6				−.1145			140	Poor	Excellent	Good
8.0	19.0				−.1318			140	Fair	Excellent	Good
2.6								170	Very good	Good	Poor
7.0		600[8]	.015					170	Good	Good	Fair
20.2	14.7	600[8]	.015		−.0690	4.4	2.23	170	Fair	Fair	Fair
27.7								170	Poor	Poor	Good
7.1				Neg				680	Good	Very good	Poor
21.6	20.8	330	.061	Neg	−.0952		2.45	680	Fair[9]	Good	Fair
33.8	18.1	370	.067	Neg	−.0978	5.2	2.56	680	Very poor	Good	Very good
31.0	14.0			Neg	−.1012			680	Very poor	Good	Very good
37.6	17.9			Neg	−.0935		2.32	680	Very poor	Fair	Very good
9.6								380	Good	Very good	Fair
21.5	20.1	410	.007		−.1029		2.51	380	Very good	Very good	Good
32.6	17.1	600[8]	.009		−.1043	6.0	2.39	380	Good	Good	Good
29.7	20.0	600[8]			−.1079		1.73	380	Good	Good	Very good
35.9	14.3				−.1018		2.60	380	Fair	Fair	Very good
17.6	27.4		.061	Neg	−.1422			290	Very good	Very good	Good
20.4	27.9		.067	Neg	−.1635			290	Very good	Very good	Very good
22.5	22.3			Neg	−.1510			290	Very good	Good	Very good
9.3	36.1				−.2320			350	Very good	Very good	Good
20.9	26.3				−.2318		2.14	350	Fair	Very good	Good
19.7	29.4				−.2036[7]			350	Fair	Very good	Very good
10.5	31.5	30			−.2125				Poor	Excellent	Poor
13.2								140	Fair	Excellent	Good
17.1	26.4				−.1117		1.72	140	Good	Excellent	Good
2.6				Some[10]				99	Fair	Excellent	Very poor
5.7	52.8	130	.152	Some[10]	−.1189[7]			99	Good	Excellent	Poor
13.5	34.2	330	.164	Some[10]	−.1612	5.3		99	Good	Excellent	Poor
16.7	35.3			Some[10]	−.1542			99	Good	Excellent	Very good
22.5	20.7			Some[10]	−.1592			99	Good	Excellent	Good
3.3	35.0								Good	Excellent	Good
13.9	46.4				−.1525				Good	Excellent	Good
17.7	34.3				−.1660				Good	Excellent	Very good
4.5	38.3	150	.089		−.1030			14	Fair	Very good	Very poor
8.0	35.4	180	.102		−.1200	5.3		14	Good	Very good	Poor
9.5	32.5				−.1238			14	Poor	Excellent	Poor
13.0					−.1482			14	Very good	Very good	Fair

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Continued from p. 593

TABLE 2—LIGHTWEIGHT CONCRETE

Material	Cement content sacks/cu yd	Water content lb/cu yd	Slump in.	Unit wt fresh concrete lb/cu ft	Vinsol resin grams/cu yd	Comp. strength psi: 7-day fog	7-day fog 21-day 50 percent humidity	7-day, fog 21-day 50 percent humidity, 14-day 122 F	Flexural strength psi 180-day	E x 10⁻⁶ 7-day fog, 21-day 50 percent humidity, 14-day 122 F	Unit wt cured conc, lb/cu ft 42-day
Perlite Sample No. 5	5.74	997	2.0	67.1	85.5	135	340	285		0.12	36.6
Exfoliated vermiculite Sample No. 1	5.24	827	4.5	59.7	80.4	150	221	270	125	[9]	32.9
	7.39	938	3.5	73.3	81.9	325	400	465	250	0.24	49.8
Exfoliated vermiculite Sample No. 2[11]	3.16	503	7.0	35.9		25	[9]	40		[9]	23.2
	4.95	619	3.9	47.8		65	130	140	115	[9]	31.1
	6.88	637	4.0	55.0		185	265	340	160	[9]	36.4
	9.24	635	5.0	62.0		375	515	620	360	0.23	50.3
Diatomaceous earth	2.94	1054	3.5	78.6		85	140	130		[9]	46.9
	4.95	993	3.5	83.2		220	305	[9]		[9]	55.4
	7.00	901	4.0	86.7		240	330	320	40	[9]	60.0
	8.93	951	4.0	90.8		405	485	470	45	[9]	65.4

[1] Static modulus of elasticity (6 x 12-in. cylinders)
[2] Failed by 25 percent weight loss or by breaking apart.
[3] Expansion at 310 cycles. (0.07 percent expansion at 310 cycles is defined as failure.)
[4] BTU per in. per sq ft per hr per deg. F (medium temperature range).
[5] Average psi for 1-in. consolidation.
[6] 28-day values.

TABLE 3—CEMENT CONTENT, UNIT WEIGHT, AND THERMAL CONDUCTIVITY FROM 200

Aggregate	200 psi: Sacks cement	Unit wt lb/cu ft	*k* factor	500 psi: Sacks cement	Unit wt lb/cu ft	*k* factor	1000 psi: Sacks cement	Unit wt lb/cu ft	*k* factor
Sand and gravel (control)	[1]			1.5[1]	134.0	6.00	2.0	134.5	6.00
Expanded Shale Sample 1	[1]			1.8[1]	75.0	2.85	2.6	77.5	2.93
Sample 2	[1]			2.2	87.5	3.45	3.4	90.0	3.65
Pumice Sample 1	2.5	69.0	1.90	3.0	71.5	1.95	3.8	74.0	2.05
Sample 2	2.5	72.0	1.95	3.0	74.0	2.00	3.8	77.0	2.10
Sample 3	2.8	65.0		3.4	67.5		4.4	70.5	
Sample 4	3.8	64.0	1.75	4.5	67.0	1.85	5.5	70.5	1.95
Pumicite	2.8	65.0		3.2	68.5		7.0	80.0	
Scoria	2.8	73.0	1.70	3.8	76.5	1.80	5.5	82.0	2.00
Slag Sample 1	2.5	72.0	2.05	3.0	76.0	2.10	4.4	81.0	2.27
Granulated No. 2	5.2	65.0		6.0	68.0		9.4	79.5	
Granulated No. 3	4.0	76.0		5.0	78.5		[2]		
Perlite Sample 1	3.0	32.0	1.25	4.8	42.0	1.45	7.3	53.0	1.77
Sample 2	5.0	41.0		6.8	48.5		8.3	55.5	
Sample 3	5.0	34.0		6.8	43.5		9.0	54.5	
Sample 4	5.6	51.0		7.5	68.5		10.0	64.5	
Sample 5	6.0	38.0		8.0	48.0		[2]		
Vermiculite Sample 1	4.5	29.0	0.80	7.6	42.5	1.30	[2]		
Sample 2	5.5	33.0	0.95	8.5	46.0	1.40	[2]		
Diatomaceous earth	5.0	55.0		10.0	68.0		[2]		

Note: Values shown in this table were interpolated from test data shown in Table 2.
Unit weights are for dry concrete.
[1] Insufficient amount of cement to produce workability.
[2] Not possible to obtain this strength using 100 percent of the indicated aggregate.

MIX DATA AND TEST RESULTS—Concluded

Comp. strength wt ratio psi/lb	Absorption percent by dry wt	Freezing & thawing: Cycles to failure[2]	Wetting & drying: Expansion percent[3]	Alkali expansion 12-mo	Drying shrinkage percent 80-day	Thermal coeff. of expansion x 10^6	Thermal conductivity k[4]	Aggregate crushing strength[5]	Nailability	Sawability	Workability
17.7	81.7								Fair	Excellent	Very poor
8.3	84.4				−.2279[7]			14	Poor	Excellent	Good
9.3	55.4				−.2042[7]		1.76	14	Fair	Excellent	Good
1.8				Neg				14	Very poor	Excellent	Fair
4.4	75.0	600[8]	.076	Neg	−.2658		0.85	14	Poor	Excellent	Good
9.4	30.7	600[8]	.056	Neg	−.2440[7]	7.9	1.15	14	Fair	Excellent	Very good
12.4	21.5			Neg	−.3120		1.50	14	Fair	Excellent	Very good
2.8				Some[10]				240	Poor	Excellent	Fair
		10	.080	Some[10]	−.0465[6]			240	Poor	Very good	Fair
5.3	61.8	10	.203	Some[10]	−.3186[7]	6.3		240	Fair	Very good	Fair
7.2	38.1			Some[10]	−.0452[6]			240	Good	Good	Fair

[7] 42-day values.
[8] Test discontinued before failure.
[9] Unable to obtain test result (specimen too weak or fractured in handling).
[10] Expansion is not due to reaction between alkalies in cement and aggregate.
[11] Manufactured with air-entraining agent added to the material.

(k) OF LIGHTWEIGHT CONCRETES FOR COMPRESSIVE STRENGTHS RANGING TO 5000 PSI

2000 psi			3000 psi			4000 psi			5000 psi		
Sacks cement	Unit wt lb/cu ft	k factor	Sacks cement	Unit wt lb/cu ft	k factor	Sacks cement	Unit wt lb/cu ft	k factor	Sacks cement	Unit wt lb/cu ft	k factor
2.8	135.5	6.10	3.8	136.0	6.20	4.9	137.5	6.50	6.3	140.0	7.00
3.6	80.0	3.05	4.7	83.0	3.18	5.8	86.5	3.32	7.4	91.0	3.50
4.3	92.0	3.85	5.4	94.5	3.90	6.5	97.0	4.00	8.2	100.5	4.20
5.3	78.5	2.20	7.5	84.5	2.50	[2]					
5.3	82.5	2.30	7.5	88.5	2.55	[2]					
8.3	81.0		[2]								
[2]											
7.6	89.0	2.25	9.2	94.5	2.45	[2]					
6.5	88.5	2.55	10.2	101.0	3.00	[2]					
[2]											
[2]											
[2]											
[2]											

aggregates tested show distress in the test specimens resulting from drying shrinkage.

The thermal coefficient of expansion values for the specimens tested were approximately the same as those obtained on regular concrete, averaging 5.2×10^{-6} with the exception of diatomaceous earth and exfoliated vermiculite which had coefficients of 6.3 and 7.9×10^{-6}, respectively.

Freezing-and-thawing and wetting-and-drying durability tests were performed on aggregate samples selected from each type. The majority of these were found to have surprisingly good resistance to the forces of these tests, and compare favorably with normal sand and gravel concrete. Of all the lightweight concretes tested containing 7 sacks of cement per cu yd, only 3 were classed as below good durability measured by freezing and thawing. These were samples of very fine material containing no purposefully entrained air, including diatomaceous earth, perlite, and finely divided pumicite. The concrete made with the vermiculite aggregate gained weight and expanded through absorption of water during the freezing-and-thawing tests. However, this concrete showed no signs of breakdown after 600 cycles. This good resistance can be attributed in part to the comparatively high air content which resulted from the air-entraining agent included in the vermiculite as received. In the wetting-and-drying test only the diatomaceous earth, perlites, and vermiculites, showed more expansion than would be expected with normal concrete.

From petrographic analyses it was determined that all aggregate types considered in the investigation, with the exception of expanded shale and clay and scoria, contained components which might produce deleterious expansion when combined with high-alkali cements. Despite the fact that many of the lightweight concretes were composed of highly reactive materials, none of the expansion obtained on bars made from the various aggregate types can be attributed directly to alkali-aggregate reaction. Substantial expansion with all cements, regardless of alkali content, was produced in the concrete made with diatomaceous earth and perlite. Vermiculite was not tested in this test which used 1 x 1 x 10-in. bars stored in moist storage because it was shown by petrographic analysis to have no reactive constituents. It is known that highly reactive siliceous materials, when used in sufficient quantities, will not produce disruptive expansion. Such materials, when finely ground, are being used as pozzolanic additives to counteract expansive reaction, and it is believed that in the case of lightweight-aggregate concrete the pessimum proportions of siliceous material to alkalies is greatly exceeded and expansion due to this cause is not indicated.

The ease with which lightweight concrete can be sawed or nailed is an important consideration in its use as a building material. It was

found that the majority of the lightweight concretes tested could be readily cut with an ordinary hacksaw with the exception of expanded shale and clay concretes. Treated pumice and scoria concretes were classed as having fair sawing properties, though they could be sawed only with considerable time and effort. All of the other lightweight concretes were found to be quite adaptable to sawing with little effort and time involved. With the exception of the expanded shale and clay and treated pumice, nails could be driven into all concretes tested but were not held firmly enough by all to be classed as good nailing concrete. Slags, natural pumice, perlites, and scoria could be classed as the best nailing concrete of the group tested and were the only ones that nails could be readily driven into and held firmly enough to be adaptable to most construction purposes.

CONCLUSIONS AND RECOMMENDATIONS

The following conclusions are based on tests previously discussed.

1. The strength of lightweight concrete is dependent on the strength of the aggregate particles and the richness of the mix, but in general no amount of cement will produce concretes having strengths above 1000 psi for concretes weighing less than 50 lb per cu ft or above 2000 psi for concretes weighing less than 80 lb per cu ft, dry weight. The relationship between strength and density is shown in Fig. 1.

2. The thermal conductivity of concrete decreases as the weight of the concrete is decreased. This property of the concrete seems to be influenced more by the density of the concrete than by the characteristics of the aggregate (Fig. 2).

3. The lightweight concretes having strengths above 2000 psi all had suitable resistance in the freezing-and-thawing tests and had comparatively low drying shrinkage. These concretes should be completely suitable for all types of concrete construction where heavy weight is not a requirement.

4. Disruption resulting from alkali-aggregate reaction is not indicated in lightweight-aggregate concrete.

5. Air entrainment is recommended for lightweight-aggregate concrete to improve workability and reduce separation during placing. The slump should be maintained at less than $2\frac{1}{2}$ in. wherever possible.

6. Additional tests should be made to (a) correlate the properties of dry tamped lightweight concrete with those of plastic concrete mixes, (b) determine the bond strength of lightweight concrete to reinforcing steel, (c) investigate the flow of lightweight concrete under load and cracking tendencies of large slabs, (d) investigate the use of various admixtures and wetting agents, (e) determine the most practical grad-

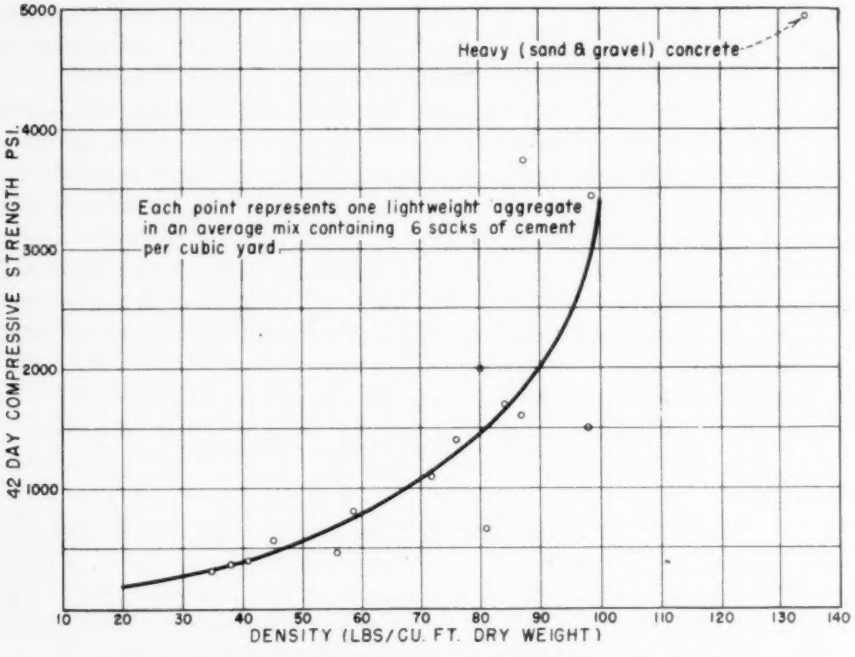
Heavy (sand & gravel) concrete
Each point represents one lightweight aggregate in an average mix containing 6 sacks of cement per cubic yard
42 DAY COMPRESSIVE STRENGTH PSI.
5000
4000
3000
2000
1000
10 20 30 40 50 60 70 80 90 100 110 120 130 140
DENSITY (LBS/CU. FT. DRY WEIGHT)

Fig. 1—Strength of lightweight concrete is a function of its density

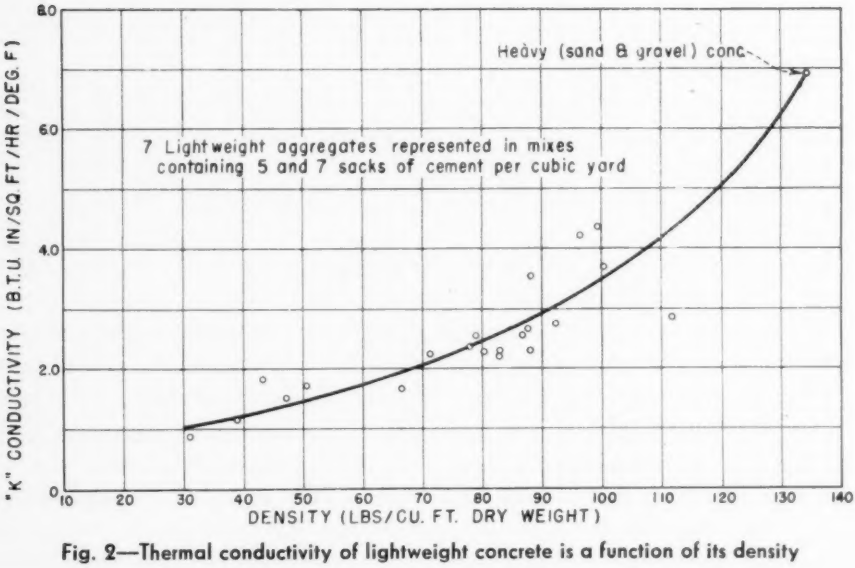
Heavy (sand & gravel) conc.
7 Lightweight aggregates represented in mixes containing 5 and 7 sacks of cement per cubic yard
"K" CONDUCTIVITY (B.T.U. IN/SQ FT/HR/DEG. F)
8.0
6.0
4.0
2.0
0
10 20 30 40 50 60 70 80 90 100 110 120 130 140
DENSITY (LBS/CU. FT. DRY WEIGHT)

Fig. 2—Thermal conductivity of lightweight concrete is a function of its density

ings for various aggregates, and (f) investigate comparative heat resistance properties of various lightweight concretes.

ACKNOWLEDGMENT

The authors are indebted to M. E. Hickey, Engineer, Bureau of Reclamation, Denver, Colorado, for his assistance in compiling the test data and tables included with this paper, and to the laboratory personnel who conducted the tests.

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Title No. 45-35

A summarization of published information on air-entraining concrete and the effect of entrained air on the properties of concrete.

What Have We Learned About Air-Entraining Concrete?*

By J. F. BARBEE†

SYNOPSIS

A review of ACI published data on air-entraining concrete, this paper summarizes methods of control and the effect of air entrainment on durability, plasticity and workability, strength, resistance to abrasion and other properties of concrete. The effects of variations in cement and water content, sand-coarse aggregate ratio, mixing time and the use of calcium chloride as an admixture are described. Changes in technique necessary to successfully place, vibrate and finish air-entraining concrete are pointed out. A detailed bibliography follows.

GENERAL

Entrainment of air in portland cement concrete is considered one of the outstanding developments in concrete technology in recent years. The addition of entrained air to normal concrete results in better workability, making placement easier and in greater durability when exposed to severe weathering and to chemicals used for ice control. Both of these contributions have long been needed and should result in concrete of much longer life and of greatly reduced maintenance cost.

In recent years a number of articles pertaining to air-entraining concrete have been published in the ACI Journal.‡ This paper summarizes information available in the Journal at the time of writing (April 1948). Data from other sources and from Journal papers of a later date may give rise to questions concerning some of the conclusions reported herein. Information is so presented that the general concrete technician can more readily use the available data in his daily concrete practice. Unfortunately, data covering field control are rather limited.

*Presented at the ACI 45th annual convention, New York, N. Y., Feb. 23, 1949. Title No. 45-35 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 8, Apr. 1949, *Proceedings*, V. 45. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Aug. 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

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‡See bibliography at end of text.

The purposeful entrainment of air in concrete can be accomplished in two ways: by the use of air-entraining portland cement, that is, cement in which the air-entraining agent has been interground during manufacture and by adding the air-entraining agent directly to the concrete at the time of mixing. As a result, a new type of concrete mixture has been developed, one which, in addition to the cement, water, and fine and coarse aggregates generally used, now has a fifth ingredient, purposefully entrained air.

Air entrainment in concrete is most satisfactorily accomplished when the air-entraining action of the agent is not dependent upon chemical reaction with the cement. Minute disconnected air bubbles formed by the agent lubricate the concrete by their ball-bearing effect on the mix. Generally, the degree of the resulting increased lubrication varies with the amount and type of the agent used, the grading and physical properties of the fine aggregate and the type and time of mixing. The addition of air to a concrete mix in general makes possible a reduction in the amount of water and fine aggregate required for placeability. It improves the workability, increases the durability and decreases the permeability and the strength of the concrete at any given water-cement ratio.[1] The entrained air tends to make the concrete more homogeneous and does not have any apparent material effect on the concrete's thermal properties.[2]

AIR-ENTRAINING AGENTS

Materials which have been used for the purpose of entraining air in concrete include: natural wood resins (such as Vinsol resin), animal or vegetable fats and oils and their fatty acids, various wetting agents, water-soluble soaps of resin acids and other miscellaneous materials.[3] More data are available on neutralized Vinsol resin and on Darex as air-entraining agents than on any others. In the remainder of this paper, it may be assumed that unless otherwise stated, the air-entrainment referred to has been obtained by the use of one or the other of these two products.

Vinsol resin is the agent most generally interground with the cement during manufacture to produce air-entraining cement. Since the entrainment of air in concrete is most satisfactorily accomplished when the agent used does not react chemically with the cement it is now required to be added in a neutralized form. The quantity added is that necessary to produce a cement which will meet the A.S.T.M. air-entraining requirements in the mortar test. Air-entraining cements meeting the current A.S.T.M. specifications generally entrain a satisfactory amount of air in concrete with cement contents ranging from 4½ to 6½ sacks per cu yd. Since variations in grading, type and time of mixing, slump, etc.,

affect the final air content of concrete, many authorities prefer to add the air-entraining agent in the field.

EFFECTIVE AIR CONTENT

Tests indicate that the quantity of air entrained in a given concrete mix depends upon several variables, including the type and grading of aggregate, the cement content of the mix, the brand of cement, the plasticity and consistency of the mix, the type of mixing, the mixing time and the type and amount of air-entraining agent used.[4a] In a given concrete mix, the amount of entrained air increases with an increase in the sand-coarse aggregate ratio and with increased slump or flow of the mixture.[5] As the cement content of the concrete is increased, greater amounts of air-entraining agents are required to produce a given air content.[3] This may be due in part to the decrease in fine aggregate required. Since the percentage of air entrained appears to be a function of the quantity of fine aggregate present in the mix, it follows that the optimum percentage of air ought to be based on the amount of fine aggregate present in the concrete mix rather than on a fixed percentage of the total mixture.[5]

The amount of air entrained in normal cement concrete, generally accepted as averaging about 1 percent, apparently varies from approximately 0.8 to 2.7 percent over a fairly wide range of mixtures, water contents and mix proportions.[6] A small increase in air content over that obtained with normal portland cement greatly improves the resistance of concrete to freezing and thawing. Thus, concrete having an air content of about 3 percent has markedly improved durability.[3]

The amount of air entrained in a concrete mix can probably be most practicably controlled by varying the amount of admixture used. The percent of entrained air can be controlled to some degree by varying the cement or sand content of the mix, the time and type of mixing, or the consistency of the mix. However, since the available mixing, transporting and placing equipment on a job generally is not subject to change, variation in the amount of agent used provides the most practical method of control of the amount of air entrained.[5]

For concrete ranging in cement content from 4½ to 6½ sacks per cu yd a maximum increase in durability with no appreciable reduction in strength appears to be reached when the air content is approximately 4 percent. This percentage is considered to be about the optimum for use in concrete mixtures containing coarse aggregate up to about 1½-in. maximum size.[5]

DURABILITY OF AIR-ENTRAINING CONCRETE

A marked increase in durability is indicated by laboratory tests, amounting in some cases to several hundred percent. Field observations

of air-entraining concrete pavements confirm the available laboratory data.

The increased air content of the concrete produced by the air-entraining agent, rather than the agent itself, probably is the factor most responsible for the greatly improved durability, since when entrained air is removed from freshly mixed concrete all improvement in the durability of the resultant concrete is lost.[1f,3,7]

Apparently the minute dispersed air bubbles provide expansion chambers, which will relieve the pressure created in the concrete due to volume change caused by changes in temperature and moisture and by the expansion due to the formation of ice crystals. The reduced water-cement ratio and reduction in the number of capillaries in the mortar caused by bleeding probably also contribute to the increased durability.

Laboratory tests indicate that between 400 to 600 billion air voids are entrained in a single cu yd of concrete containing 3 to 6 percent entrained air.[8] The air voids are predominantly less than 50 microns but range considerably in size.

Entrained air is not a cure-all for all failures of concrete due to lack of durability. For example, appreciably less improvement is noted in the durability of concrete containing certain aggregates known to produce concrete of poor durability.[1o] The entrained air does, however, apparently retard the destructive action of materials of questionable quality.

PLASTICITY AND WORKABILITY

Entrained air acts as a lubricant of high value, making the concrete more plastic and workable and less subject to bleeding than similar concrete made without air entrainment. Entrained air tends to reduce segregation of water, fine aggregate and coarse aggregate in a mix, and when this does occur, such segregation is generally less serious. One exception is that segregation of fine aggregate is more likely to occur when the amount of sand in the mix is in excess of the optimum. Entrained air increases the cohesiveness of the mix, aids in retarding settling of the aggregate during transportation and increases the capacity of the concrete for plastic deformation.[9]

When air is entrained in a concrete mix and the slump is held constant through reduction of water and cement, the workability is found to be better than that of the concrete without entrained air.[4a] A given percentage of entrained air in lean mixes produces a greater improvement in workability and plasticity than is found with mixes of higher cement content. This permits an appreciably greater reduction in the water-cement ratio of the leaner mixes for any given slump.[7]

CONCRETE STRENGTH

Air entrained in concrete without any other changes being made in the mix reduces both compressive and flexural strength. These reductions in strength have been and can be minimized or often eliminated by proper changes in mix design. These changes should consist of a reduction in both the water-cement ratio and the sand cement. Such changes in lean mixes (4 sacks per cu yd or less), often result in concrete of higher strengths than the corresponding normal concrete.[1f] When the cement is kept constant and the water-cement ratio and sand content are reduced, as permitted by the introduction of entrained air, the effect on the strength varies from a slight increase for mixes of low cement content to a decrease of 3 to 5 percent in strength for each percent of air entrained for mixes of higher cement content.[4d]

ABRASION RESISTANCE OF AIR-ENTRAINING CONCRETE

Abrasion resistance of concrete containing less than 6 percent air has been found to differ little from that of normal cement concrete when the cement contents of the comparative concretes are constant.[4c] However, there is progressive increase in the abrasion loss as the air content increases. When the amount of entrained air approaches 10 percent, abrasion resistance is markedly low, even though the concrete would be extremely resistant to freezing and thawing. Since concrete used in pavements is generally specified to have from 3 to 6 percent entrained air, its abrasion resistance should be satisfactory. However, in hydraulic structures which are subject to erosion, it is suggested that the minimum percentage of air required for durability be specified at not to exceed 5 percent.[10]

MIXES OF HIGH CEMENT CONTENT

It has been said that very little, if any, improvement in durability as indicated by resistance to freezing-and-thawing tests may be expected with concrete of high cement content regardless of the use of air-entraining agents. Apparently there is a critical cement content in excess of 7 sacks per cu yd, beyond which little improvement in durability can be obtained by the use of entrained air.[7] This is a controversial point, however, and more data are needed. It seems probable that if sufficient air is entrained, an improvement in durability will be noted, regardless of the richness of the mix. The resistance to freezing and thawing of hardened concrete containing less than approximately 7 sacks per cu yd, is markedly improved by the addition of entrained air, this improvement varying inversely with cement content.[12]

For good results with concrete mixes of higher cement content containing relatively small coarse aggregate, more air should probably

be entrained than would be required in relatively lean mixtures containing larger coarse aggregate.[5] The sand content of the richer mixes is generally much less than of lean mixtures, and the optimum percentage of entrained air apparently is a function of the quantity of the fine aggregate rather than a fixed percentage of the total mixture.

WATER CONTENT

Where the consistency of a given mix is held constant, increases in air content accompany decreases in water content, the increase in air being directly proportional to the decrease in water. This reduction in the water-cement ratio, permitted by entrained air, to maintain desired workability results in a basic increase in the strength and durability. In general, it appears that the lower the cement content of the mix, the greater is the possible reduction in water requirement per unit increase of air entrained. The slump of air-entraining concrete is apparently more sensitive to changes in water content than is the slump of non-air-entraining concrete. Air-entraining concretes having a slump of 2 in. or more are more workable and placeable than non-air-entraining concrete of the same slump.

When the water content of an air-entraining concrete mix is such that the consistency as indicated by the slump becomes extremely high, the air content decreases.

SAND-COARSE AGGREGATE RATIO

The introduction of entrained air into an otherwise normal concrete mix results in a marked increase in the volume of the mortar component. To maintain the same slump and cement factor, it is necessary to reduce both the quantity of mixing water and the quantity of fine aggregate in the mix.[1f] Reductions of fine aggregate to compensate for the increased fatness of the concrete caused by air entrainment must be governed by the placeability of the concrete.[3]

An increase in the sand-coarse aggregate ratio results in an increase in the air content of the mix. Available data indicate that the coarse aggregate itself has little effect, except as its affects the amount of sand required in the mix.

Air-entraining concrete mixes designed in the laboratory for field use must often be adjusted under field conditions since these conditions usually produce either a greater or less air content than that obtained in the laboratory.[1h]

CALCIUM CHLORIDE INTEGRAL ADMIXTURE

An integral admixture of calcium chloride (2 percent by weight of cement) will increase air entrained by concrete from approximately

0.8 percent to 1.5 percent.[6] When calcium chloride is used integrally, the strength of air-entraining concrete is increased in about the same ratio as when it is used in regular portland cement concrete. In amounts up to 2 percent, a calcium chloride admixture either dry or in solution, increases early strength without decreasing the effectiveness of the air-entraining agent.[3] Air-entraining concrete, whether it contains calcium chloride or not, will still be much more resistant than normal concrete to scaling and to freezing and thawing. The use of calcium chloride in air-entraining concrete mixes does not affect the increase in workability of the concrete resulting from the entrained air. Apparently its only effect is to accelerate the setting time and to permit the concrete to be finished more promptly.[1n] The use of calcium chloride to increase early strength at low temperatures is probably more beneficial with air-entraining concrete since low temperatures retard strength gain in air-entraining concrete slightly more than in normal concrete.

Very few data are available on the more commonly used admixtures other than calcium chloride and fly ash. The addition of fly ash in amounts up to and including 15 percent does not appreciably affect the percent of air entrained in the concrete.[6]

EFFECT OF MIXING TIME

When the agent is not dependent on reaction with the alkalies in the cement for its air-entraining property, the maximum amount of air is entrained after a fairly short mixing time (approximately 5 minutes in small tilting mixers), after which the amount of air decreases as the mixing time increases.[5] Since prolonged mixing in either transit, paver or stationary mixers decreases the air content, it is necessary, when using an air-entraining admixture, to add enough to obtain the required amount of air at the end of the mixing period. In using air-entraining cements, the mixing operations should be controlled to obtain the required amount of air. Data are lacking concerning the effect of mixing time on air content for various types of mixers.

BLEEDING AND PAVEMENT FINISHING

Air-entraining concrete in the plastic state is much more easily placed and has a marked tendency to bleed less than comparable ordinary concrete.[1o] This reduction in bleeding reduces the flotation of laitance upward and results in less separation of the mortar from the under side of coarse aggregate particles.[5]

The reduction in bleeding is so great that in concrete pavement construction the rate of bleeding often is below the rate of evaporation.[12] When this occurs the pavement surface becomes dry and sticky, making

it necessary to shorten the time between placing and finishing operations. With air-entraining concrete, the finishing operations usually can be started and completed sooner than with non-air-entraining concrete. In hot weather, final finishing operations must follow closely behind the finishing machines since there is very little free water on the surface for lubrication.[1n]

VIBRATION OF AIR-ENTRAINING CONCRETE

Vibration is an effective means of consolidating air-entraining concrete. However, any method used to consolidate such concrete should be applied only to the extent required for the needed compaction.[14] Generally, more care is required with air-entraining concrete than with normal concrete, the vibration generally resulting in a slight decrease in air content. This slight decrease may be due to the removal of large air bubbles which are incidentally entrapped in the concrete, as in concrete without entrained air.

There is a danger that in prolonged vibration some of the entrained air may be removed. Some investigators have found that 50 percent of the entrained air in concrete has been lost when the vibration was continued for 2½ minutes, and that as high as 80 percent of the entrained air was lost when vibration was continued for 9 minutes.[15] It is believed, however, that the loss in air will not be appreciable if vibration is moderate and only to the extent necessary to obtain the required compaction.

Since the concrete generally placed with vibration should have a fairly low slump, damage may be done if the amount of vibration is not sufficient to obtain the consolidation required. When air-entraining concrete is placed with vibration, the sample used to determine the air content should be taken, whenever practical, from the form after vibration.[16] The amount of agent added may thus be varied to provide the desired percentage of air after vibration, and as much vibration applied as is necessary for good compaction. When air-entraining cement is used the amount of entrained air may be regulated to some degree as previously described.

EFFECT OF CONCRETE TEMPERATURE ON THE AMOUNT OF AIR ENTRAINED

The amount of air entrained in a concrete of given mixture and consistency decreases as the temperature of the concrete increases. Tests indicate that in concrete at 50 F, the amount of air will be approximately 130 percent and, at 100 F, about 77 percent of that at 70 F.[4d] The rate at which the entrained air decreases with an increase of temperature is greater within the range of temperatures (60 to 80 F) normally encountered in the field. The decrease in the air content per unit increase in

temperature is apparently more marked as the slump of the concrete increases.[17]

EFFECT OF AIR-ENTRAINMENT ON OTHER CHARACTERISTICS OF CONCRETE

Modulus of elasticity

Available data indicate that the modulus of elasticity of concrete mixes having the same water-cement ratio and the same aggregate combinations is reduced from 2 to 3 percent for each percent of air entrained in the concrete.[3,4a]

Sulfate resistance

The few data currently available on the effect of air entrainment on the sulfate resistance of concrete indicate that when the cement factor is not less than 5½ sacks per cu yd, the entrainment of air increases the resistance. This is particularly true as the C_3A content of the cement decreases.[4o]

Bond to steel

Test data indicate that in some cases the bond to steel is reduced with the entrainment of air in the concrete mix.[13] However, this reduction may be minimized as are the reductions in compressive and flexural strength by maintaining the air content of the concrete at the optimum.[5,12]

Pumping

Air-entraining concrete has been handled and placed by pumping with generally good results. The increased workability caused by air-entrainment is considered an aid to the movement of concrete by this method. The only deviation from normal pumping reactions has been a somewhat more pronounced evidence of internal expansion as the concrete reaches the end of a long pipe line. It appears that the same variations in pumping experienced with changes in normal cement concrete mixtures may be expected with air-entraining concrete.[1m]

Volume changes due to moisture and temperature changes

Changes in volume of concrete due to moisture and temperature changes are apparently not materially affected by the entrainment of the optimum amount of air.[2,5]

DETERMINATION OF AIR CONTENT

For proper control of air-entraining concrete the amount of air in the mixture must be known. Among several methods by which the air content may be accurately determined are the rolling, the pressure gage, the hook gage and the gravimetric.[4h,k,17] All give reasonably satisfactory results when used strictly in accord with the outlined procedure.

It is recommended that one of such methods be used to determine the air content.

CONCLUSION

On reviewing the effects of air entrainment on the various properties of concrete, it becomes apparent that far better results may be obtained when the proper amount of air is entrained in the concrete.

Apparently, further study is necessary to determine just what is the optimum amount of air in concrete for various purposes. For highway work, 4 percent has been advanced as the optimum. Additional data must be forthcoming to determine if this is the case. Optimum air contents for other work must also be established. It may be that based on the mortar component of the mix there is an optimum air content suitable for all work.

ACKNOWLEDGMENT

The author is indebted to R. R. Litehiser, Chief Engineer, and E. W. Cummins, Engineer, of the Testing and Research Laboratory, Ohio Department of Highways, for their assistance in the preparation of this paper.

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The production of concrete masonry units has doubled in the past decade. New and enlarged plants, improved machinery and new techniques strive to meet the challenge of present demand.

Developments in the Manufacture and Technology of Concrete Masonry Units*

By JAY C. EHLE†

SYNOPSIS

The author touches the high spots of developments in the manufacture of concrete masonry units listing the many problems which now face the manufacturer. Rapid growth of the industry in the past ten years and the change from tamping to vibration methods is described. New plants are notable for labor saving equipment compared with those of the middle 1930's. In addition to block manufacture and several methods of curing, in use and proposed, the author mentions briefly: block-handling equipment, maintenance and repair of the plant, and the greater supervisory skill now required for successful operation.

INTRODUCTION

During the past 10 years the production of concrete masonry units has approximately doubled. This growth has been both a cause and an effect of developments in their manufacture and technology. Plants have been enlarged, new plants established; new and more efficient equipment has replaced the old every few years; labor saving methods have been installed and production processes speeded up—all in an effort to keep output abreast of demand. The capital investment in the concrete block industry is about 650 million dollars—this estimate based on plant investment of $65,000 per million block annual capacity and 1 billion block per year total output. At the present rate of production of approximately 1 billion 8x8x16-in. block equivalents plus sills, lintels, and other collateral precast masonry units, the industry's gross sales probably exceeds 200 million dollars per year.

To maintain low manufacturing costs and selling prices the industry has spent large sums to mechanize its production operation. The modern plant is designed and equipment is engineered for maximum efficiency consistent with requisite product quality.

*Presented at the 45th ACI annual convention, New York, N. Y., February 25, 1949. Title No. 45-36 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 8, Apr. 1949, *Proceedings* V. 45. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Aug. 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

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The typical products plant of ten years ago had tamper-type machines, volume batching, manual methods of handling all kinds of material including the blocks themselves, the rail car system, inadequate steaming setups, and uninsulated curing rooms. New machinery and methods have made all these obsolete.

Discard of old curing methods began with the use of lightweight aggregates. Volume batching went out also as the use of lightweight aggregates increased because they require greater skill in mixing and proportioning. These are a few reasons why the increased use of lightweight aggregates alone had a strong effect on the concrete products industry. Other major factors are the increase in building and the great reduction in the use of clay tile as a structural unit. (In Cleveland in 1939 ninety-five percent of the hollow masonry was clay tile; five years later the percentages were reversed—95 percent were concrete units.

MATERIALS STORAGE AND BATCHING

Most modern products plants have overhead bins for storing aggregates and bulk cements. The ability to stockpile cement in sufficient quantity during the cement shortage is extremely important and has saved many a modern plant from shutting down. A modern plant should have bin storage sufficient for at least three days to a week, depending on the way cement is delivered. Overhead bins are usually fed by belt conveyors or bucket elevators. Occasionally where the terrain is favorable trucks dump directly into the bins from a ramp. This unfortunately is unusual.

Below the bins an up-to-date plant has automatic weighing and batching equipment to reduce both manpower and human errors. The traveling weigh batcher—in use for some time—has been in recent years made automatic and now the cement and even the aggregates can be controlled by the push of a button or two. Constant care is still required, however, especially with lightweight aggregates where the bulking factor due to moisture gradation fluctuates constantly.

MIXING EQUIPMENT

When the tamper machine went out and the high production block machine of the 1940-1946 era came in, a change in mixing equipment was required. New and larger mixers were developed. Ten years ago a 28-cu ft mixer was considered about standard. This mixer could supply a machine producing 300-400 blocks per hour, and give sufficient mixing time. About 1940 the 42-cu ft mixer became the popular size and in 1947 the 50-cu ft mixer became necessary.

The batch type mixer is now most commonly used with an average mixing time of 5 minutes as a minimum for most aggregates. It is almost trouble free and does a good job if not overloaded.

With the new high-speed block machines mixing is more of a problem as the speed is double that of the block machines of a year or two ago. Proper supply of these machines requires a 100-cu ft mixer (much too large for good usage), two 50-cu ft mixers emptying in the same chute, or a continuous mixer which has a large capacity but requires little space.

The continuous mixer which has a long open drum and a shaft carrying many steel blades was in use 20 years ago but fell into disfavor because the materials could not be accurately proportioned. Another objection to the continuous mixer when using a lightweight aggregate was that the water could not be added to the aggregate before the cement as aggregate and cement were fed into it simultaneously. Water should be added first to give better mix results. Its mixing action was good—in a matter of a minute the concrete was more thoroughly mixed than in a batch mixer after several minutes. It was almost trouble free and required much less power than a batch mixer.

During the last year considerable work was done with this type of mixer. Now the water can be added to the aggregate before the cement. This, and other improvements have changed the picture in regard to the continuous mixer. It has more capacity, cuts power and labor (under ideal conditions no operator is required) and it does a good job of mixing.

BLOCK MAKING MACHINERY

The main part of any plant is the block machine. Most machines now used are of the vibrating type. There were several reasons for changing from the tamping machine in addition to labor saving and cost reduction. Again the advent and increased use of lightweight aggregates is an important factor. The tamping machine crushed the aggregate, thus losing part of its benefits. With the development of the vibrating machine this problem vanished and many plants changed immediately to this type of machine with its other advantages of greater speed and lower labor costs in addition to producing a better product. The Wisconsin tests published in the ACI JOURNAL in 1939* were also a great influence in the change from the tamping to vibration.

All these modern machines are fully automatic, requiring but one man to operate the air hoist in the front of the machine which transfers the blocks to the racks. The pallets on which the blocks are formed are now returned to the machine automatically, or rather without effort, by the same air hoist and magnets. In short, the modern high production machine is a rather complicated but reasonably trouble-free piece of

*Wendt, Kurt F. and Woodworth, Paul M., "Tests on Concrete Masonry Units Using Tamping and Vibration Molding Methods," ACI JOURNAL, Nov. 1949, *Proc.* V. 36, pp. 121-164.

equipment which makes a good product at the rate of 600 to 900 per hour with a minimum of labor. The development will not stop there; in the near future the "super-duper" product machines will be in use.

At least two machinery manufacturers are building, or in the process of designing, machines which will produce 1200-1800 blocks per hour. One machine is now being built which will make six 8 x8x16-in. blocks on one large pallet instead of the usual three. Another large company is coming out with a machine with an entirely different type of vibration based on the theory of resonance. This will do away with the old off-center-shaft rotary vibration and will produce nearly vertical vibration of the mold. The manufacturer obtains in excess of 1200-1400 blocks per hour with two reciprocating mold boxes—really two machines in one. There seems to be nothing in the near future which will replace vibration as the medium of compaction.

BLOCK HANDLING EQUIPMENT

Probably the greatest single step in plant simplification was the adoption of the lift truck. Prior to 1938 all blocks were handled by rail cars or by hand from the machines to the steam rooms, thence to the yard piles and finally to trucks or railroad cars. About 1938, the platform truck came into use and by 1943 was almost standard. This truck successfully handled racks any place in the plant or yard. Spring supported platforms eliminated the impact when picking up a rack. About 1944 the fork-lift truck began to replace the platform truck rapidly as it can do all the platform truck could plus stacking the cubed blocks to almost any height desired. An important function is the loading of trucks with cubes from the yard, simplifying immensely the loading problems. In eight hours one fork-lift truck can load 350 tons of lightweight blocks or about twenty-four thousand 8x8x16-in. equivalents. Formerly this same task would have required 8 to 10 good men.

The fork-lift truck is a veritable "jack-of-all-trades." It can be equipped with a scoop for loading loose materials, a blade for light bulldozing or a fork for moving unit loads. It can also be used as a platform for many jobs such as cleaning light fixtures or painting ceilings.

Two of the unsolved problems in block handling are the forming of blocks into cubes and unloading blocks from trucks at the job site. Considerable work has been done on both problems. Several companies have made yard hoists for the removal of blocks from racks but on the whole they have been unsuccessful. The most successful job-site method uses an overhead crane mounted on the truck bed to unload the cubes of blocks. In most cases, however, good labor is more economical. Much more work should and is being done on both these problems.

CURING

The most controversial subject in the concrete products field is that of curing and the equipment that goes with it. Naturally, the best way to cure any concrete is by warmth and moisture followed by drying. Concrete so treated should be nearly stable. This whole process in a concrete products plant must be reduced from weeks to hours to syncronize with the manufacturing process. The block producer has been working toward accomplishing this in a period of twenty-four hours. In the writer's opinion it has not been done excepting by high-pressure steam curing, nor will it be done for a while. However, some plants are approaching this or at least are doing good jobs in three to seven days.

Modern plants are using either high-pressure boilers, low-pressure boilers or steam generators but it is the writer's belief that the generally accepted practice is the use of a low-pressure boiler operating at 5-15 psi depending on the curing layout. The piping layout should be as short as possible and conform to the best steam fitting practice.

The technology of curing is still under debate but from the numerous tests some facts are known. Additional moisture is needed for good curing of the molded block, heat accompanying this moisture is advantageous, and the method of injecting this heat and moisture is important. Curing rooms should be insulated and sealed as tightly as possible.

In one popular new method of curing, racks of blocks are placed in a room and left undisturbed for 1½-2 hours after the room is loaded and sealed to allow the concrete to get its preliminary hardening. During this period the heat of hydration of the cement raises the temperature a few degrees depending on the outside temperature and the insulation of the room. Live steam is then admitted at a boiler pressure of 8-15 psi, depending on the size of the curing chamber. This steam passes through a jet designed to give a nozzle velocity and capacity which will allow the steam to reach the far end of the room without impinging on either the side walls or the roof. If the steam strikes these colder surfaces some of its moisture is lost for curing. When the steam vapor reaches the end of the chamber it turns down and passes through the racks and along the floor back to the point of origin. In this way a circulation of moisture-laden vapor is obtained. The steam is allowed to enter the room and raise its temperature at a rate not to exceed one degree per minute. When the internal temperature of the blocks reaches that of the atmosphere in the steam room the steam is turned off and the blocks are allowed to soak in the vapor for several hours. Then the remaining vapor is exhausted from the room by suction fans, the room is opened and the blocks are taken to the stockpile for further drying.

The critical point with this type of curing is the temperature at which the steam is to be turned off. This is determined by suspending a green

block from a gram scale and weighing it every few minutes. It has been found that the freshly molded block gains in weight up to the point where the internal temperature equals that of the surrounding air. Thermocouples and a potentiometer record the temperature of the block and surrounding air. When these two temperatures meet the vapor pressure inside the block exceeds that on the outside and any further temperature rise causes the block to lose moisture and hence to some extent retards the hydration of the cement. This critical temperature varies with the curing layout and the specific heat of the aggregates and may be as high as 190 F or as low as 150 F. It has been found that a too rapid rise of temperature in a freshly molded block may cause it to craze and crack but with a preliminary set a rise of even one degree per minute has no effect on the concrete.

It is possible with this type of curing to develop about 80 percent of the ultimate strength of block within 24 hours and usually, in most climates, practically all of its strength in 14 days.

Another problem in connection with conventional curing processes which is gradually being solved is the drying of the blocks after completion of curing cycle. Some manufacturers prefer to store the blocks under covered sheds long enough to allow the concrete to lose moisture down to 40 percent of its total absorption. This method requires a great deal of shed room and yard space. Many manufacturers are working on dryers to remove this moisture in the curing chambers at the end of the curing and soaking period. This is now being done by circulating hot air but there is considerable work being done with direct-fired units and desiccants. If a simple method using dessicants can be devised it probably will provide a more natural drying cycle.

High pressure steam curing has had a limited use for a number of years. It consists of curing blocks in high pressure vessels at about 125 psi and temperatures in the vicinity of 350-360 F. It does a good job but is very expensive to install and operate and tends to make some aggregates very brittle. However, it delivers a dry block with approximately its 28-day strength in 8-10 hours.

QUALITY CONTROL

The segregation of aggregates in the bins presents a troublesome problem to plants using fairly dry aggregates or which often must let their bins run low. Other plants where aggregates are moist and in uniform supply, experience little difficulty. Most plants now are trying to break the aggregate down into two sizes, usually from $\frac{3}{8}$-$\frac{1}{4}$ in. and $\frac{1}{4}$ in. to dust. Other plants are planning to go one step further and make a three-way grading and to use skip grading when practical.

A modern plant using a weigh batcher for proportioning its aggregates usually takes unit weights several times a day and corrects the scale for whatever variation occurs. This is an important step in quality control and results, with all other things equal, in the finished blocks having like-textured faces and equal strength.

One of the troubles with block concretes is that in the main (especially with lightweight aggregate) they are harsh and unworkable. With no slump and the available water sufficient only to assure hydration of the cement it is a double problem to make a mix workable without increasing the weight of the block or increasing the water to a point where it will smear the side of the block or cause it to slump under vibration. Gradation and air-entraining cements seem to help and a combination of them appears to be the answer. At present time there is no sure way to get the proper consistency of mix other than by sight. Most aggregates contain varying amounts of water and except with dry aggregates a water meter is of little use. The addition of even an extra gallon of water to a 50-cu ft batch sometimes results in an unworkable mix. The water added to a mix sometimes varies over a day's time from nothing to 4 or 5 gal. per sack of cement.

The modern plants of today are rapidly adding instruments and testing equipment and some of their laboratories rival and surpass those of commercial organizations. Many plants now have their own testing machines, test curing, roll-tap sieves, and other equipment. Potentiometers and temperature recorders are common. Many of the larger plants have their own plant engineer. All these have made the plant owners more quality conscious and there is a daily check on strength, moisture content, and absorption. A great many of these plants use the recognized commercial laboratories for check testing, and for published tests required by local and state regulations.

MAINTENANCE

As modern machinery is put into operation another change in concrete products manufacturing comes into being. From the simple maintenance practices of a few years ago the block producer finds himself confronted with a much more complicated procedure requiring a separate department rather than something to be done at odd times.

The maintenance foreman should have more than a passing knowledge of electricity, boilers, and mechanics. The plant should have a machine shop equipped with at least a lathe, drill press, hydraulic press, grinders and possibly a milling machine. This is especially true in locations not served by well-equipped private shops.

Modern plants should have electric welders operated by competent men. In recent years the use of hard surface welding rods has cut down

repairs and comes under the heading of preventive maintenance. Plants that crush their own aggregates use it for facing crushers. Other plants use it for lining mixers and preventing wear on lift forks. Often the life of a machine can be increased several times, resulting in material savings. Heat treating and hard chroming of parts—especially mold boxes—has resulted in material saved and a better product.

Lift trucks and delivery equipment must be placed on a regular greasing and overhauling schedule. Oils and greases must be carefully selected for the right jobs. In short, maintenance of the machines, mixers and other plant equipment must be placed on the same level of importance as manufacturing. A ten-minute breakdown due to lack of maintenance now can mean the loss of several hundred blocks.

PRODUCT DESIGN

In the years past there has been some study of concrete block design. For a number of years the standard three-core block has been the most popular. This block is satisfactory as far as ease of cutting on the job is concerned. The mason can split it in many ways by cutting at the core openings. In the opinion of many engineers and manufacturers, however, it is not as strong structurally as the two-core block. Many plants have now switched to either a three-core block with a smaller center core giving it greater resistance to tensile stresses or to a two-core block re-designed for greater strength. This two-core design still has the drawback that it cannot be cut satisfactorily in the field but the modern block machine can make ¼, ½ and ¾ lengths and masons are being educated to order these special sizes when starting a job.

There are many newly designed specials such as pilaster blocks, lintel blocks, soffit blocks, sill blocks and others which have come into use in the last few years and have meant many hours and dollars saved by the mason contractor. With the modular block and specials many jobs are actually laid out and constructed with no job cutting. One of the effects of the use of specials is to increase the plant investment in molds and to increase change-over costs from one size to another.

CONCLUSION

The modern plant of today is no longer inexpensive. A one-machine plant may cost in excess of one hundred thousand dollars with the larger three and four-machine plants costing approximately a half million dollars. These plants, however, have low manufacturing costs and produce a better quality block than their less modern competitors. The success of the block industry has been in plant modernization, high production, and constant improvement of the finished product. It is the hope of all concerned that this progress will continue.

Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

Recommended practices for sampling and testing ready-mixed concrete

NATIONAL READY MIXED CONCRETE ASSN., Washington, D. C., 20 pp.

These recommended practices summarize and interpret A.S.T.M. Standards of particular interest to the ready-mixed concrete industry. A secondary purpose is to supplement them in the light of experiences of representative producers and testing engineers.

A laboratory investigation of mineralogical, chemical and physical properties of limestone aggregates

J. L. YOUNG, JR. and J. H. HAVENS, Highway Research Board Paper, 28th annual meeting, December 1948 — HIGHWAY RESEARCH ABSTRACTS

This is an investigation of intrinsic properties of limestone aggregates to determine their effect on the behavior of concrete and bituminous mixes made with these aggregates. Data presented deal only with determination of intrinsic properties and not with tests for behavior of the mixes, which are in progress.

Effects of various admixtures on air entrainment, water requirement and compressive strength of concrete

DELMAR L. BLOEM, *Misc. Publication* No. 22, National Ready Mixed Concrete Assn., 10 pp.

This paper discusses the effects of 13 different admixtures on air entrainment, water requirements and compressive strength of concrete. All concrete tested was of medium richness and consistency, carefully designed and controlled to produce valid comparisons among the various admixtures.

Portland cement concrete resurfacing on an old concrete pavement in Georgia

W. F. ABERCROMBIE, Highway Research Board Paper, 28th annual meeting, December 1948 — HIGHWAY RESEARCH ABSTRACTS

In 1947 the Georgia State Highway Department resurfaced a 4.1 mile section of portland cement concrete pavement which had been in place almost 25 years. As a considerable portion of the old road was on desirable grades and still in reasonably good condition as much of the old pavement as possible was utilized by building a concrete resurface. The contractor widened, resurfaced and constructed new sections in one continuous operation.

*A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 8, Apr. 1949 *Proceedings*, V. 45. Address 7400 Second Boulevard, Detroit 2, Mich.

Cooling towers at West Ham

Concrete and Constructional Engineering, V. 43, No. 12, Dec. 1948, pp. 377-380

Reviewed by GLENN MURPHY

Two of three hyperbolic reinforced concrete cooling towers are under construction by the British Electricity Authority at West Ham. Each tower is designed to cool 2,800,000 gal of water per hour through 12 F, and has a height of 288 ft above the level of the pond incorporated into its base. The structure is supported on concrete piles about 25 ft long. The shell of the tower, which is from 5 to 16 in. in thickness, is supported on 72 diagonal columns, each 14 in. square. Timber forms are used.

British standard code of practice for the structural use of normal reinforced concrete in buildings

The Codes of Practice Committee, British Standards Institution, London, 1948.

BUILDING SCIENCE ABSTRACTS

The eight sections of this code, representing good present-day British practice, are as follows: general; materials, appliances and components; design considerations (basic principles, beams and slabs, columns, solid reinforced concrete flat slab construction, walls, fire resistance); work off site; workmanship; inspection and testing; maintenance and appendixes (tests on concrete, etc.). Tables of design data are included.

Application of ultrasonics to the testing of concrete

R. JONES, *Research*, V. No. 8, 1948, pp. 383-384

CERAMIC ABSTRACTS

The assessment of the properties of concrete by determination of the natural frequency of transverse vibration is restricted to specimens of a certain shape. Determinations of longitudinal wave velocity in laboratory specimens by measurement of the time taken for a short-duration ultrasonic disturbance to be propagated through a known length of concrete between transmitting and receiving piezocrystal gauges at frequencies between 100 and 250 kc./sec. showed that it is possible to detect systematic differences between specimens. The development of internal cracks due to frost damage can be followed.

Current design of concrete pavement in New Jersey

WILLIAM VAN BREEMEN, Highway Research Board Paper, 28th annual meeting, December 1948

HIGHWAY RESEARCH ABSTRACTS

This paper pertains to the design of concrete pavement currently employed in New Jersey in the construction of highways to carry heavy truck traffic. The design features described and discussed are: pavement thickness, slab length and width, reinforcing steel, dowelling of expansion joints, joint filler, and sub-base construction. Information is given in regard to the performance of expansion joints employed in the past. The effects of corrosion of dowels and reasons why the design includes expansion joints are discussed. Included are: traffic data, the results of load-transfer tests on various sizes, shapes, and lengths of dowels and the results of tests in connection with the sliding resistance of concrete slabs on various subgrade materials.

Prestressed reinforced concrete

R. H. EVANS, *Institute of Municipal Engineering Journal*, V. 74, No. 11, pp. 565-596

BUILDING SCIENCE ABSTRACTS

The fundamental theory governing the even and the uneven prestressing of reinforcement in concrete is briefly presented and is followed by a general discussion of loss of prestress due to shrinking, and of strain and creep of the concrete, with particular reference to prestressed reinforced concrete beams. The properties of typical pre-

stressed concrete members are described, as well as some practical examples of their application to structures, *i.e.* beams, sleepers, transmission poles, pipes, water tanks, pressure tunnels, floors and bridges.

What do we know about gradation of aggregates in concrete? (Ou en est la granulometrie du beton)

A. Joisel, Reprinted by the Laboratories du Batiment et des Travaux Publics, Paris
from *La Technique Moderne—Construction*, V. III, No. 3, June, 1948 Reviewed by Ivan M. Viest

The author reviews a number of theoretical and empirical methods for determining the gradation of concrete aggregates. He discusses first the theories of Feret, Abrams and Caquot at some length. Several practical solutions of the gradation problem are presented next, including grain size curves as proposed by various persons, and a number of empirical and geometrical methods. Finally, the relation of gradation to the overall problem of making concrete is considered. The paper includes a short bibliography.

Vibrations of concrete and reinforced concrete (Vibration du beton et du beton arme) (with summaries in English and German)

R. Dutron, International Association Bridge and Structural Engineers,
Preliminary Publication, 3rd Congress, 1948, pp. 173-186 Applied Mechanics Reviews
Oct. 1948 (Durelli)

This is a report of results obtained from several vibration tests of concrete. Radius of action of the internal vibrator as a function of plasticity of the concrete and of duration of the vibration, and volumes of concrete influenced by superficial vibration as a function of plasticity and grading are studied in particular. The influence of these variables on compressive strength, density, and porosity is given in charts and diagrams. The author also gives some other qualitative results and advice regarding good practice.

The theory and practice of constructional design

G. Torda, Paris, 1948 (Dunod). 385 pp. Building Science Abstracts

To simplify and speed up design procedure for structures involving the use of many separate members and elements, the author has developed a generalized method for determining the lines of influence. This enables the effect of the live and dead loads, both concentrated and distributed, to be evaluated by means of equations of influence. These equations are established in a convenient form and their numerical values tabulated for a whole range of structural members including beams, frames, brackets, arches, etc. The data thus permit rapid determination of bending moments, shearing and normal stresses, etc., in plain and reinforced concrete members. The design procedures are fully illustrated by 58 numerical examples.

Experimental concrete road in Nottinghamshire

Concrete and Constuctional Engineering, V. 43, No. 12, Dec. 1948, pp. 398-401
Reviewed by Glenn Murphy

The Road Research Laboratory has recently completed the first of a number of experimental concrete roads. The object of the experiments is to determine the least thickness of slab necessary for different soils and volumes of traffic. Slabs of 3, 4, 5, 6, 7 and 8 in. are used, to cover the range from too thin to too thick. In some parts the slabs are laid on a soil-cement base from 2 to 6 in. deep, in others the slabs are cast on waterproof paper laid directly on the ground. Both plain concrete and reinforced concrete slabs are included and several arrangements of joints are under investigation. Thermocouples, a temperature recorder and a traffic counter are a part of the installation.

Use of reinforcement in concrete pavements

R. R. PHILIPPE, Highway Research Board Paper, 28th annual meeting, December 1948 — HIGHWAY RESEARCH ABSTRACTS

The author presents the results of full scale traffic tests on concrete pavements reinforced to degrees varying from normal temperature steel to full structural quantities, The tests include full scale traffic with wheel loads of 37,000, 60,000 and 150,000 lb on designs presumed to be adequate for these loads. Comparisons are made also to non-reinforced pavements tested under the same loads at the same time and at the same location. It is concluded from these tests that the economic benefits to be derived from the use of reinforcement lie in the added life of pavement after the initial break, but that the added bearing capacity due to reinforcement is of doubtful economic value.

Hydro-electric works in Scotland

Concrete and Constructional Engineering, V. 43, No. 12, Dec. 1948, pp. 365-373

Reviewed by GLENN MURPHY

This is essentially a progress report on construction of the Loch Sloy Dam, the tunnel under Ben Vorlich and the generating station at Inveruglas on the shore of Loch Lomond. This development, one of 12 projects under construction, has a rated capacity of over 130,000 kw with an operating head of 910 ft. The dam, which will contain over 200,000 cu yd of concrete is a buttress type 1160 ft long with a maximum height of 165 ft. Crushed rock for the concrete is transported from the quarry, 1½ mi. from the mixing plant, by belt conveyor. The concrete is transported from the mixer to the site by a skip on rails, and final deposition is from an aerial cableway. The 1¾-mi. tunnel is drilled through rock and is lined with concrete throughout most of the length. Near the outlet the tunnel divides into two branches each lined with a steel pipe 10 ft in diameter. A surge chamber 35 x 25 x 90 ft, with floor and walls of concrete, is incorporated into the supply system. A number of construction details are given.

Rules for the use of indented and smooth rods of high elastic limit in reinforced concrete (Regles d'utilisation des rondes creneles et lisses pour beton arme)

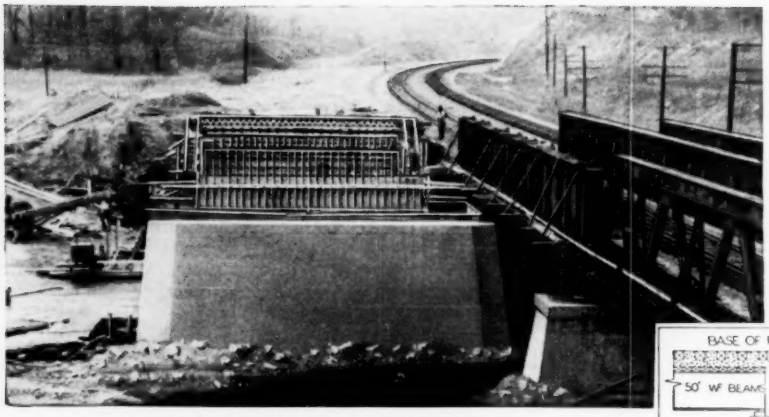
Institut Technique du Batiment et Des Travaux Public, Paris, Oct. 1948, 57 pp. 300 francs plus postage

Reinforcing rods with high elastic limit for concrete are so economical that French steel firms, urged by Monsieur Caquot and assisted by the Chambre Syndicale des Constructeurs de Ciment Armé, have undertaken to manufacture them. A large number of them will be produced with projecting fins which chiefly serve to increase bond.

These rods are already on the market and a committee of the Ministry of Reconstruction and Town Planning has been working under pressure to adapt the 1945 Code for reinforced concrete to cover this new type of reinforcement, and the revised code is now available to builders. The new code is numbered to correspond with the 1945 code and contains explanatory notes incorporated in the text. In addition, extensive appendices contain detailed illustrations showing methods of use and simplifying the calculation of normal stresses and different conditions of bond.

ACI News Letter

Vol. 20 No. 8 JOURNAL of the AMERICAN CONCRETE INSTITUTE April 1949



Scene during PREPAKT construction of Santa Fe bridge near Wilbern, Illinois.

PREPAKT REPORTS ON . . .

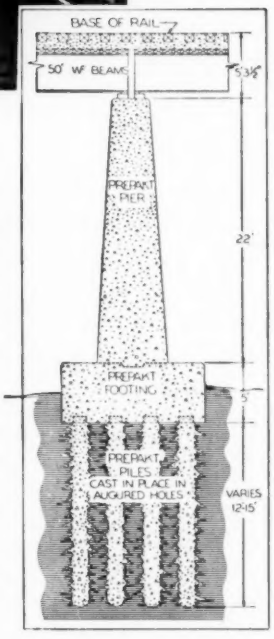
BRIDGE PIER STRUCTURE

As part of the continuous Santa Fe maintenance and modernization program, Intrusion-Prepakt recently completed the pier structures for a main line Santa Fe bridge in Illinois. Several outstanding features and methods were incorporated in the construction of the piers for this bridge.

First, the piles were cast in place in augured holes (see sketch). Extremely high load carrying capacity was provided in these piles by the double penetrating action of the coarse aggregate and natural soil penetration of PREPAKT grouting materials. These insured exceptional skin friction and high point bearing.

Next, the footings were constructed over the projecting tops of the piles, after which the pier shafts were cast on top of the footings, being "tied in" with reinforcing rods extending from the footings.

PREPAKT methods and materials thus provided these piers with exceptional strength and high resistance to deterioration from weather and other factors.



Typical Cross Section of PREPAKT pier and foundation.

HIGH RESISTANCE TO FREEZING-THAWING • LESS CRACKING
LOWER PERMEABILITY • RESISTANCE TO SULPHATE MOISTURE
HIGH COMPRESSIVE STRENGTH • HIGH BONDING STRENGTH

THE CONCRETE WITH PREPAKT EXTREME DURABILITY

INTRUSION-PREPAKT, INC. THE PREPAKT CONCRETE CO.

CHICAGO • TORONTO CLEVELAND 14, OHIO PHILADELPHIA

ACI

News Letter Contents

April 1949

On the Cover—Alpena, Mich. plant of the Huron Portland Cement Co.; world's largest single cement plant after a recent expansion. Bulk powdered materials are moved by aeration through a stationary woven canvas. Air bulking enables cement to flow on a slight incline thus eliminating many conveyors.

Abrams Aerial Survey photo

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The ACI 45th Annual Convention, New York City

Feb. 23-25, 1949

The 45th Annual Convention seemed to please the "customers." When customers are pleased the ACI workers, who spent months at the task of preparation for the event are also pleased. It was a first class meeting without breaking records numerically. Its registration of 531 (a check of all cards showed the figure of 528 published in March to err on the low side) was well above the average over the years of record and is close to past registration figures in New York. 1935—527, 1937—550, and 1939—502. A well-rounded program reaching the interests of practically all ACI membership groups held large audiences throughout the meeting. Much credit is due the Technical Activities Committee, under the Chairmanship of D. E. Parsons, for its careful, selective program planning and to the New York local committee headed by M. J. McMillan for early suggestions of program material.

Surely some of the 1949 Convention workers and still new ones now will be working on through the coming months for a Chicago success in February 1950.

Wednesday afternoon featured concrete pavement durability and air entrainment in concrete with major papers by B. D. Tallamy, Charles M. Noble, J. F. Barbee, and R. F. Blanks and W. A. Cordon.

Structural design of two-way slabs and wall and column footings and the use of experimental aids in structural design were discussed in papers on Thursday morning by C. P. Siess and N. M. Newmark; F. E. Richart; and R. E. Glover, O. J. Olsen and Carl Zangar.

Continued on p. 4

Ready-mixed concrete, and the problems in its proper control at point of delivery, had lively discussion following an introductory paper by Stanton Walker.

The popular annual research session provided brief papers by 10 authors who gave confidential reports on current research studies.

At the closing session there were two papers on architectural concrete by J. J. Hogan and E. B. Oberly; a paper on lightweight aggregate concrete by R. W. Kluge, M. M. Sparks and E. C. Tuma; and a paper on the manufacture of concrete masonry units by J. C. Ehle.

The highlight of the get-together luncheon Thursday noon was the annual presentation of awards by President Blanks. Three hundred applauded the honors and heard Francisco Gomez-Perez of Mexico invite ACI members to the First International Engineering Congress in Mexico City the first week in May.

Herbert J. Gilkey was elected president to succeed R. F. Blanks; Harry F. Thomson was elected a Vice-president; Harmer E. Davis, H. C. Delzell, Nathan M. Newmark and R. W. Spencer, Directors. (See p. 10.)

The chairmen of 13 technical committees took advantage of the convention gathering to call meetings. In general, the attendance at committee meetings was good and although not announced as open meetings a number of visitors were present at most sessions.

Many registrants availed themselves of the opportunities outlined by the local committee to visit interesting construction projects in and around New York City.

Local committee did fine job

In bringing the convention to a close President Gilkey expressed the sincere appreciation of the Board for the fine job done by the local committee in looking after the many details which make up a successful meeting. Chairman M. J. McMillan and the committee deserve much credit and the thanks of the Institute for a good job, well done. Mr. McMillan's able co-workers were: E. K. Abberley, Waldo G. Bowman, J. H. Chubb, A. Burton Cohen, Roger Corbetta, Joseph Di Stasio, Homer G. Farmer, Elliot A. Haller, Frank L. Kelly, Emil Praeger, M. A. Swayze, B. H. Wait and C. A. Willson.

Problems in Concrete Pavement Surface Durability

Wednesday Afternoon, February 23

Said President Blanks: "I now declare the 45th Annual Convention of the American Concrete Institute in session.

"It never ceases to me to be a wonder how year after year engineers and others who are interested in this drab stuff we call concrete come back for more and more. There must be something about it to have an attraction like that.

"First, I would like to observe a moment of silence for our co-worker who has left a spot that will not be filled, our late friend, Art Boase.

(A moment of silence).

"Thank you."

After the announcement of F. N. Menefee and Douglas E. McHenry as tellers to canvass the annual election ballots, Mr. Blanks turned the meeting over to Frank H. Jackson, Chairman for the afternoon's technical program.

Control of concrete pavement scaling caused by chloride salts

by B. D. Tallamy, State Superintendent of Public Works, New York State, described the evolution of an oil-spray treatment for recently laid concrete pavements which was expected to prevent scaling caused by the application of concentrated salts for ice control. He pointed out that modern use of highways has made such measures necessary and that the effects on concrete are often

severe. A field observation of reduced scaling where oil from vehicles had coated the pavements was described and the laboratory tests which followed to determine the most effective and economical oil treatment were presented in detail. Finally, the design and use of spraying equipment and an indication of the expected results were given. (Mr. Tallamy's paper was published in full in the March JOURNAL, p. 513.)

In extension of Mr. Tallamy's paper an associate, Ira Paul, N. Y. State Dept. of Public Works, presented results of preliminary laboratory tests made after the paper was written. Early results are encouraging. It is believed the oil spray acts first as a sealer on fresh concrete preventing the loss of water during curing and then penetrates into the concrete to fill or oil-coat the air voids resulting in greater resistance to freezing and thawing. Slides were shown of concrete specimens after freezing-and-thawing tests to demonstrate the effectiveness of the oil treatment.

L. E. Andrews, Regional Highway Engineer, Portland Cement Assn., in prepared discussion pointed out that where air-entraining concrete was used on New York highways it was highly successful.

Early air-entraining concrete test roads built in the period 1938-40 indicated no scaling according to a PCA survey reported in 1948. Mr. Andrews recommended that air-entrainment be specified on all contracts for pavements in severe climates.

Hubert Woods, Director of Research, Portland Cement Assn., in reading discussion prepared by H. F. Gonnerman, said that in 1936, an ACI paper by Gonnerman, Timms and Taylor described results of tests by PCA using various surface treatments to prevent scaling. Only linseed, soybean, and perilla oils were fully effective. Waste crank case oil was also tested with less satisfactory results. New York field tests being conducted this winter will be watched with interest.

Chairman Jackson called on A. G. Timms, Public Roads Administration, to show a few slides and discuss work the PRA is doing along this line.

Experience with air-entraining concrete in New Jersey

was reported by Charles M. Noble, State Highway Engineer of New Jersey. The increase in the use of air-entraining concrete from an experimental basis in 1945 to the present general use was illustrated by data on contracts let for pavements in that state. The increasing loads to which highways are subjected, higher vehicle speeds, and greater winter travel give rise to many problems in design and construction. That disintegration of highway pavements is not a problem in New Jersey was attributed to excellent aggregates, rigid laboratory controlled design of mixtures, low water-cement ratios, exacting laboratory inspection of materials at the source and on the job, strict inspection of batching and mixing equipment, rigid inspection and construction supervision on the job and use of Type II Cement. While too little time has elapsed since the general use of air-entraining concrete to form judgment, indications are that results fully justify its use. (Published March JOURNAL, p. 521).

In a prepared discussion, I. L. Tyler, Portland Cement Assn. stated that in an experimental pavement consisting of 48 sections 12 ft wide, cements made from six different clinkers were used to make concretes with and without air-entrainment. No significant difference was found between Type I and Type II cements. It was felt that a differentiation in type of cement need be made only in mass concrete or where sulfate conditions exist.

Elliot A. Haller, Haller Testing Laboratories, Inc., reported finding that when its analysis is close to the Type II range, Type I cement is about equal to Type II. That is, it has a low C_3A content.

H. S. Mattimore, Pennsylvania Dept. of Highways, reviewed the accidental

discovery of the benefits of air entrainment and observed that many of the old "accidental" air-entraining concrete pavements had excellent service records in resistance to freezing and thawing. Therefore it can be seen that air-entrainment is not new or untried. "It has a record." What is new is the name, the manufacture under control and the designation of approved A-E agents.

C. H. Cash, Michigan State Highway Dept., read discussion by C. C. Rhodes giving Michigan experience dating from 1940 when a test road using air-entraining cement was laid using Type IA cement. In 1941 air-entraining concrete was used on many jobs and since 1943 all pavements have contained entrained air. The optimum is 4 percent and with this amount scaling due to salts is avoided although raw salts are often used to eliminate bad ice conditions. In a few cases air-entraining concrete pavements laid in early morning have cracked. This condition is locally termed "morning sickness."

E. E. Bauer, Univ. of Ill., in reading a paper by H. W. Russell said that in Illinois, 3 to 5 percent of air is entrained in concrete for pavements. Type I and IA cements are generally used; Type III and IIIA in some cases. A notable increase in resistance to chloride salts has resulted.

L. E. Andrews, Portland Cement Assn., added further comment on the reduction of scaling through use of air-entraining cement.

What have we learned about air-entraining concrete?

by J. F. Barbee, Engineer, Testing and Research Laboratories, Ohio Department of Highways, reviewed information published in the ACI JOURNAL on air-entraining concrete and summarized methods of control and the effect of air entrainment on durability, plasticity and workability, strength, resistance to abrasion and other properties of concrete. The effects of variations in cement and water content, sand-coarse aggregate ratio, mixing time and the use of calcium chloride were discussed. (Published in this JOURNAL, p. 601.)

Practices, experiences and tests with air-entraining agents

was presented by R. F. Blanks, chief, Research and Geology Division, U. S. Bureau of Reclamation. Co-author was W. A. Cordon, Materials Engineer in the same organization. Mr. Blanks discussed the advantages of air entrainment with regard to durability, permeability, workability, reduction in alkali expansion, time saving, reduction in water and cement, temperature rise, strength, abrasion resistance, monolithic lightweight concrete and mass concrete. U. S. Bureau of Reclamation experiences were discussed with regard to pumping, transporting and placing, grading of aggregates, and factors affecting the amount of air entrained. (Published February JOURNAL, p. 469.)

The papers on air entrainment by Messrs. Barbee and Blanks prompted discussion on several aspects of the use of air-entraining cements. William Van Breemen, New Jersey, asked if calcium chloride had an adverse effect when used in air-entraining concrete. Mr. Blanks stated that the U. S. Bureau of Reclamation had used $CaCl_2$ with an A-E agent and found that it worked about the same as it did for plain concrete.

The question of the merits of a ground-in air-entraining agent *vs.* one added at the mixer was raised by J. J. White of Parsons, Brinckerhoff, Hall and MacDonald. Mr. Barbee replied that practices varied, some authorities adding at the mixer for better control. The Ohio Dept. of Highways permits both methods with no difference in results under proper controls.

Ira Paul pointed out that all factors affecting the mix must be considered in determining air in a mixture and cited an instance in an experimental pavement incorporating an asphaltic cut-back oil, RC-2, consisting of 75 percent asphalt residue and 25 percent gasoline. The concrete showed an air content of 1 percent in the laboratory and 3.5 percent in the field. This was explained by the

fact that the laboratory specimen had set around long enough for the gasoline to evaporate before the air determination was made. In the field the determination was made while the volatile gas was still in the mix and measured in the meter.

E. M. Brickett, Dewey and Almy Chemical Co., Cambridge, Mass., suggested that the quick hardening of the surface of air-entraining concrete mentioned by Mr. Noble could be attributed to the elimination of bleeding, with the result that water contained in the mass does not migrate to the surface; the surface stiffens but cannot be said to set during finishing.

The correctness of crediting air entrainment with imparting durability to concrete was questioned by Henry Comack on the basis of a loss of air when the concrete hardens. This was answered by a recommendation to adjust the air in the mix to give the final amount desired in the placed concrete. Decreasing slump and cement factor were offered as other means to increase strength and durability. Mr. Comack referred to the use of zero slump concrete by the Germans and others as a desirable practice for adoption in this country.

Chairman Jackson remarked that the desire in this country for speed and efficiency coupled with the high cost of labor and relatively low cost of materials made such practice impractical here whereas in other countries where labor might be cheap and materials expensive the more laborious processes could be used.

Henry Gollomb mentioned that by the use of the vacuum process zero slump could be, and is, produced in this country.

In reply to a question by Ray Young, Charles R. Watts Co., Seattle, as to the effects of air entrainment on the reaction to alkalies, it was stated that data indicated that with air entrainment there is no appreciable reduction in expansion by use of air and therefore it would have little effect in improving concretes of this type.

W. T. Neelands, Savannah District, Corps of Engineers, asked whether specifications for air entrainment should vary with the use of the particular part of the structure with the thought that air content for dam and spillway concrete ought to be lower, but at what figure.

Mr. Blanks indicated that since the change in resistance to erosion through air entrainment is so small little is to be gained by specifying different amounts of air in the concrete for different parts of the structure.

Structural Design

Thursday Morning, February 24

Vice President Herbert J. Gilkey presided at this session devoted to research affecting design of concrete structures.

Rational analysis and design of two-way concrete slabs

was briefed (published Dec. JOURNAL, p. 272) by Chester P. Siess, Special Research Assistant Professor of Theoretical and Applied Mechanics, University of Illinois. Co-author was Nathan M. Newmark, Research Professor of Civil Engineering at the same University.

A new method for the design of two-way building slabs was proposed and its step-by-step development was described. Several variables were studied in the analyses and included: the ratio of sides, the effect of discontinuous edges, the torsional stiffness of the beam, various types of loading, and combinations of panels of various sizes and shapes. Conclusions were stated regarding the types of loading to be considered and the values of beam torsional stiffness to be assumed in the development of the design procedure.

The design method was endorsed by Committee 321—Design of Reinforced Concrete Slabs.

Professor Newmark presented a written discussion by Mounir Badir, graduate student of the University of Illinois, in which a number of experimental structural computations were summarized.

Discussion by Jacob J. Creskoff brought out the fact that the analytical solution was based in part on experimental work. While the data are limited, all that are available were analyzed and considered. The bases of modifications were experience, a few test loads, and good judgment.

Hans Bleich, Hardesty and Hanover, New York, believed that the design method could not be applied well to bridge slabs with large concentrated loads. Professor Newmark replied that no data were given in the design method for large concentrated loads and thus it does not apply to bridge and other slabs loaded in this manner.

It was W. H. Wheeler's opinion that not enough attention was being paid to what happens in concrete and too much to what happens in the steel in the matter of redistribution of stress with time. He believed that too much steel was used for negative reinforcement and not enough for positive reinforcement.

Reinforced concrete wall and column footings

In introducing Frank E. Richart, Research Professor of Engineering Materials, Univ. of Ill., Chairman Gilkey termed his work the "first basic research in footings in many years." This paper, greatly briefed in presentation (see ACI JOURNAL, Oct. and Nov. 1948), reported an experimental investigation of reinforced concrete footings. It represents the first extensive study of the subject since the well-known work of Talbot in 1913. In the meantime there have been many developments in materials, as well as in design methods. Developments from the tests show (a) definite proof that the tensile and bond resistance depend upon the bending moment and shear found by statics by consideration of the full applied load, and not 85 percent thereof, as assumed in current building codes; (b) hooked ends of bars showed no particular advantage in bond resistance over straight bars, particularly when deformed bars of new improved types were used; (c) hooked bars produced very little effect on resistance to diagonal tension, much less than is commonly assumed; (d) welded mats proved particularly effective in resisting end slip of bars; (e) footing caps or piers are effective in reducing the amount of reinforcement and in increasing the load capacity of footings; and (f) diagonal tension seems to be the point of weakness in current design practice. The factor of safety of thin footings in this respect appears greater than in thick footings, and is generally greater in rectangular than in square footings when the conventional methods of computation are used.

In these tests, major emphasis was placed on the isolated column footing. Principal attention was given to the resistance of footings to failure by bond, diagonal tension and tension in the steel. Test footings were designed to produce these various types of failure. In addition, studies were made of the behavior of rectangular footings and footings having intermediate capitals or piers.

The tests featured measurements of tensile and compressive strains, deflections and slippage of bars. In some of the tests three grades of concrete were used, in others, five types of reinforcing bars were employed. In most of the tests, the footings were 7 ft square. They were supported on a bed of steel springs and tested in a 3-million-lb testing machine.

Professor Richart presented and interpreted the test results, but did not offer recommendations for changes in design procedures at this time.

According to Prof. C. J. Posey, State Univ. of Iowa, in special situations such as sloped footings, hooked reinforcing bars might be of value where shear is involved. A feeling was expressed that a $4D$ hook was not large enough for some purposes. Professor Richart made clear

that his findings did *not* indicate that hooks never should be used. There is some value—about 15 percent—with hooks in shear. They will continue to be used in special situations.

John R. Nichols remarked that variations in soil pressure under a slab with the direction of pressure upward and toward the center may result in more pressure under the center of the slab than at the edges making conditions not identical with the test set-up in which the slab was supported on springs which developed an equal reaction on the underside of the slab.

Experimental aids in structural concrete design

was presented by Robert E. Glover. O. J. Olsen and Carl Zangar were co-authors. All are members of the staff of U. S. Bureau of Reclamation of Denver, Colorado.

Statically determinate and indeterminate structures are generally designed by analytical methods. This paper described several experimental aids which are used in the design and stress analysis of statically indeterminate structures at the laboratories of the Bureau of Reclamation. These methods are used where analytical procedures can not be applied conveniently. Photoelastic methods using both the polariscope and interferometer, the Beggs Deformeter, Stresscoat and electric SR-4 strain gages were described and illustrated by slides. Applications for each method were given. The paper was published in full in the February ACI JOURNAL.

The use of Anders Bull's brass rod models was suggested by Prof. J. A. McCarthy, Notre Dame Univ., as being more economical, and the models easier to make than those made with some of the materials described.

SR-4 strain gages were preferred over brittle lacquer by T. B. Rights, Johns-Manville Corp. It was agreed by Dr. Glover that this might be better on large models permitting the use of the gages, but that on small models brittle lacquer was preferred.

Get-Together Luncheon, Annual Awards

Thursday Noon, February 24

More than 300 members and guests attended the get-together luncheon Thursday noon. In a fast-moving program President Blanks presented the Leonard C. Wason Medal for the "most meritorious paper" to Frank H. Jackson and Harold Allen for their report of an inspection tour in Germany,. "Concrete Pavements on the German Autobahnen," published in the June 1948 JOURNAL. The tour covered 1000 miles of the German super-highway system in the British and American Zones. The 1947 condition of the 7-year old pavements was illustrated in detail as were construction equipment and methods. Data were included on soil conditions, climate, and traffic conditions, and recommendations were made for research on paving practices in this country.

The Leonard C. Wason Medal for "noteworthy research" went to Richard C. Mielenz, Kenneth T. Greene, and Elton J. Benton for work reported in "Chemical Test for Reactivity of Aggregates with Cement Alkalies; Chemical Processes in Cement-Aggregate Reaction." At the U. S. Bureau of Reclamation Laboratories a chemical test was developed which determined the reaction of aggregates with high-alkali cements. By this procedure the prediction of behavior of aggregates in concrete provides a means to avoid the use of questionable materials.

Raymond E. Davis, E. Clinton Jansen and W. T. Neelands won the ACI Construction Practice Award for a "paper of outstanding merit" on concrete construction practice entitled "Restoration of Barker Dam" which appeared in the April

1948 JOURNAL. A unique method of stabilizing and restoring a 37-year old dam was described. A blanket of Prepakt concrete contained in a permanent form of precast concrete slabs was bonded to the upstream face of the dam when the reservoir was nearly full of water.

Francisco Gomez-Perez, representing the National Society of Professional Civil Engineers of Mexico, invited members of ACI to attend the First International Civil Engineering Congress to be held in Mexico City April 30-May 7, 1949.

New Officers, President's Address, Ready-Mixed Concrete, New Standard

Thursday Afternoon, February 24

F. N. Menefee presented the report of the tellers naming the following new officers: President (1-yr. term)—Herbert J. Gilkey, Professor and Head of the Dept. of Theoretical and Applied Mechanics, Iowa State College, Ames, Iowa; Vice President (2-yr. term)—Harry F. Thomson, Consulting Engineer, St. Louis, Mo.; Directors (3-yr. term)—Harmer E. Davis, Acting Director, Institute of Transportation and Traffic Engineering, Univ. of Calif., Berkley, Calif.; H. C. Delzell, Managing Director, Concrete Reinforcing Steel Inst., Chicago, Ill.; Nathan M. Newmark, Research Professor of Structural Engineering, Univ. of Ill., Urbana, Ill.; R. W. Spencer, Chief Civil Engineer, Southern California Edison Co., Los Angeles, Calif. Henry L. Kennedy, Cambridge, Mass. and Paul W. Norton, Boston, Mass. were appointed by the Board of Direction to fill the vacancies left by the death of A. J. Boase and the election of Mr. Thomson as vice president.

The five elected members of the nominating committee are Franklin R. McMillan, Raymond E. Davis, Roy W. Crum, Frank E. Richart and Morton O. Withey.

A petition for a change in the first sentence, second paragraph of Sec. 13, Art. II of the By-laws from "The Vice Presidents, each in the order of his institute membership seniority shall discharge the duties of the President, in his absence" to "The Vice Presidents, each in the order of his seniority in that office, shall discharge the duties of the President, in his absence," was declared carried on a voice vote subject to confirmation by letter ballot.

Retiring President Blanks then did some "concrete crystal gazing" (reported in detail on p. 565) as to what the future of concrete and concrete engineers might be. Fanciful as his talk seemed, who knows what lies before us? In a jovial air and with the following words he passed the responsibility of his office to his successor:

"And now, to use some of your own language, Herb, remorselessly, unsympathetically, and with malicious intent I take pleasure in trading places with you behind this 8-ball."

President Gilkey appropriately thanked his predecessor for an 8-ball cleverly mounted on a concrete base and assumed the chair to turn to the more serious business of the afternoon's technical program.

Controlling the uniformity of ready-mixed concrete

by Stanton Walker, Director of Engineering, National Ready Mixed Concrete Assn., Washington, D. C. Mr. Walker's discussion of control was confined to those operations peculiar to central mixing and transit mixing. The purpose was to arouse discussion rather than to lay down recommended practices. He directed attention to the mutual responsibility of producers and users in bringing about effective control. Current specifications and standards applying to the industry were mentioned briefly. (Mr. Walker's paper appears in this JOURNAL, p. 569.)

The ensuing discussion was searching, critical and constructive—the best discussion of the convention. Most outstanding remarks were for the need for sincere cooperation of the vendor, con-

tractor, owner and testing laboratory; effective rules and inspection to prevent the addition of extra water to the mix at the job; difficulties from the practical application of material testing control when the vendor is serving many jobs.

Alexander Foster, Jr., Warner Co., Philadelphia, discussed experiences of his firm with ready-mixed concrete.

S. J. Warberg, Giffels and Vallet, Inc., Detroit, held that legal and financial responsibility should be written into specifications for concrete work while others felt that despite such safeguards the selection of a contractor was more important than overly fine specifications.

A survey of 18 jobs using ready-mixed concrete was reported by A. T. Glassett which showed that on four, delivery of concrete was other than good and two of these were classified as bad. Both of these cases were truck-mixed concrete. Measured by strength and consistency, only one was unsatisfactory, one was fair and the others good. In general it was felt that large organizations produced the best ready-mixed concrete because larger staffs permitted specialization.

M. J. Roach argued that mixing water was the biggest problem with this type of operation which required close cooperation between producer and user. The close control of aggregate and barring of wash water from trucks were also advanced as valuable aids to quality concrete delivered on the job. The vendor must recognize the need for control and use inspection in cooperation with the consumer, not as something working against him.

A survey seeking complaints of 400 New Jersey building contractors on ready-mixed concrete was reported by Frederick C. Blanchard, representative of Building Contractors' Assn. of New Jersey He described this group as "horse and buggy" contractors handling small jobs. Major complaints were poor service and high cost. An example was cited of a 6-week job to which no ready-mixed concrete producer could supply more than 80 cu yd per day.

A common fault in stockpiling aggregates at central plants was brought out by Wilbur Carlson, James H. Herron Co., Cleveland. For uniformity it should be piled in horizontal layers rather than in cones which permit segregation.

H. G. Protze expressed his conclusion that control is valuable only when the engineer and vendor agree that control is desirable and work together for it.

Elliott Harris, Metropolitan Sand and Gravel Corp., described a method of controlling in-transit mixing:

In-transit mixing is used with each truck being checked for absence of wash water before loading. Most of the mixing water is added at the plant with the balance being carried on the truck. To determine the point en route at which mixing is started a survey is made to determine travel time and to select a starting point. A service man halts mixing in case of delay at the job.

It was recommended that operational policies for the ready-mixed concrete production be written and then adopted and used.

W. S. Elliott claimed that more faulty concrete is caused by too short than by too long a mixing period. Tests were cited showing that concrete was better up to 4-hour mixing time. While New York regulations permit no water until the job is reached, a job on the Brooklyn Bridge was supplied with concrete transit-mixed for 45 minutes. The general reaction of engineers who inspected the job was that the transit-mixed concrete was better and more workable.

It was the opinion of the E. O. Martinson, Koehring Co., that concrete producers were in a rut so far as sizes of aggregate were concerned, only two sizes being generally used. Larger aggregate could be used for greater strength; even 1½-in. aggregate could be used in many reinforced thin sections.

In regard to inspection C. R. Reid, Pennsylvania Turnpike Comm., recommended that proper measures for control

be written into specifications. After having o.k.'d materials, cement bins and aggregates stock piles were replenished, nullifying previous tests and requiring considerable extra work to retest the materials. He seeks a specification to prevent the indiscriminate mixing of materials already accepted by the consumer.

Andrew Rinn, Walter Kidde Constructors, Inc., concurred with Mr. Walker in the methods of control which should be excercised in the use of ready-mixed concrete. He stated that, until five or six years ago the Kidde Company preferred to mix its own concrete, but now they find jobs where it is just as economical to use ready-mixed concrete and sometimes cheaper. He agreed with Mr. Roach that wash water should not be permitted on the mixing truck; also that for short hauls, ½ to ¾ hours, there might be an advantage in central-mixed concrete.

T. B. Rights, structural engineer, Johns-Manville Corp., made a final plea for competent inspection on the job. A lot of firms for economy (?) (Mr. Right's question mark) reasons do not employ inspectors but spend good money for a good design, have good control of mixes and then through lack of inspection permit the job foreman to add a lot of water to get a wet, sloppy mix.

Portland cement paint

The report of Committee 616—Recommended Practice for the Application of Paint to Concrete Surfaces, was presented by R. E. Copeland in the absence of Chairman G. E. Burnett and approved as a standard by a vote on the floor. According to procedure for adoption of standards the action of the convention will now be referred to Institute membership for ratification by letter ballot.

Research

Friday Morning, February 25

The annual open meeting of Committee 115—Research, with Chairman Stephen J. Chamberlin and Secretary George W. Washa presiding, produced information on bond tests with air-entraining concrete; distribution of bond in embedded reinforcement; investigation of prestressed spirally reinforced columns; use of electric strain gages inside concrete members; the effect of internal temperature variation; shrinkage and plastic flow in causing modification of stress distribution near the computed position of contraflexure in the interior spans of continuous reinforced concrete slab bridges; effect of restraint and durability characteristics on concrete aggregates; effect of rate of loading on stress-strain curves of concrete beyond the ultimate strength; protective and decorative coatings for concrete masonry structures; influence of aggregates on durability of mass concrete and a method for determining the air content of hardened concrete.

Participating contributors were K. P. Billner, Vacuum Concrete Corp.; A. E. Fatin, Univ. of Michigan; W. C. Hansen, Universal Atlas Cement Co.; Eivind Hognestad, Univ. of Illinois; M. J. Holley, Jr., Massachusetts Institute of Technology; T. B. Kennedy, Waterways Experiment Station; D. W. Lewis, Purdue Univ.; R. M. Mains, Cornell Univ., H. S. Meissner, U. S. Bureau of Reclamation, whose report was presented by R. C. Mielenz; J. N. Thompson, Univ. of Texas; and L. E. Vandegrift, Ohio State Univ. Among the discussers were: N. H. Withey, Albert Weiner, James D. McCarthy, C. H. Scholer, Fred F. Bartel, F. H. Jackson, Phil H. Ferguson, V. N. Connar, Arthur P. Clark, Carl Menzel, M. A. Swayze and Ruth Terzaghi.

The paper presented by Mr. Hansen, "A Method for Determining the Air Content of Hardened Concrete," was written by Robert P. Vellines and Thomas Ason and will appear in an early issue of the ACI JOURNAL. Others may follow from time to time as more complete data and conclusions become available.

Architectural Concrete; Lightweight Aggregates; Products Manufacture

Friday Afternoon, February 25

Design details for architectural concrete was presented by John J. Hogan, Regional Structural Engineer, Portland Cement Assn., New York, N. Y.

The skilled craftsmanship required to construct architectural concrete must be preceded by proper design by the architect and engineer. The designer must understand and appreciate the fundamental requirements of this type of construction. Mr. Hogan discussed considerations appropriate to achieving the desired texture of surface as affected by form materials selected. Ornamentation should be considered in every detail from the standpoint of practicability. Construction joints are inevitable and should be specified in logical locations. Similarly, control joints should be specified with an understanding of where the most likely cracks due to volume change will occur and the design of reinforcement should be coordinated with the location of the control joints. Slides were shown illustrating how these factors influence architectural concrete.

Construction practices for architectural concrete was presented by E. B. Oberly, Construction Superintendent, Portland Cement Assn., New York, N. Y.

Detailed plans and carefully drawn specifications executed by a contractor and superintendent with the know-how of form construction are necessary for good architectural concrete. Mr. Oberly described some of the more important construction practices to be considered to achieve the results desired. Form details and concrete quality were especially discussed.

Because of the close relationship between the Hogan and Oberly papers discussion of them was combined and elicited comment on form placing in relation to window openings as the sizes vary. It was suggested that plywood forms be lined up in a horizontal direction working out from the center of the building or section avoiding narrow bands horizontally and vertically.

To avoid unwanted cracks in pilasters it was recommended that a control joint be placed at the center of steel columns and that the steel be painted to break the bond and reinforcement be turned down in the vertical plane. The crack should then come in the thinner material of the joint.

The use of diagonal bars on re-entrant angles is not objectionable but it was pointed out that control joints still would be needed to force the formation of cracks where they are desired.

The treatment of irregular joints between form panels was considered a matter of taste with the architects. Some like the finish provided by coarse form boards, while others prefer a smoother finish. The nature and limitations of concrete should be kept in mind in striving for architectural effect.

Lightweight aggreagte concrete by Ralph W. Kluge, M. M. Sparks, and E. C. Tuma, Associate Professor Structural Engineering, Purdue University (formerly with the National Bureau of Standards); and Engineers, National Bureau of Standards, Washington, D. C., respectively, was delivered by Professor Kluge. The paper summarized tests of physical properties of 11 eastern aggregates which were studied at the National Bureau of Standards for the Housing and Home Finance Agency with a view of their possible use in home construction.

F. O. Anderegg, John B. Pierce Foundation, asked why masonry units were designed for 1200 psi when loads in walls were found to be sometimes only 20-30 psi. He suggested that a design requirement of 500-700 psi would be adequate and permit the use of materials providing better insulating and sound-deadening

properties. From the floor it was observed that the higher design requirements probably were meant to insure durability.

Developments in the manufacture and technology of concrete masonry units

by Jay C. Ehle closed the technical sessions. In the short time available he covered the high spots of the progressive status of this industry in the United States. Continuous mixers are returning to favor by eliminating interruptions to operations and permitting better scheduling and more economical use of plant. Handling of units has been simplified by use of platform and fork-lift trucks. In general, curing practice favors low pressure boilers delivering steam at 5 to 15 psi. Air-entraining cements have been used since 1942. Dessicants are being tried as aids to drying in some plants. That owners of plants are becoming more quality conscious is evident by employment of engineers in the larger plants and the willingness to adopt new product designs and more advanced machinery.

Committee Meetings

February 23 and 24, 1949

Chairmen of 13 committees report actions taken in meetings held February 23 and 24 at Hotel Statler, New York. In general, the attendance at committee meetings was good and although not announced as open meetings a number of visitors were present at most sessions.

Committee 115—Research (Executive group)

Professor S. J. Chamberlin, Chairman, called a meeting of the Executive Group to discuss recent suggestions for modification of some of that committee's procedures.

Committee 208—Bond Stress

This Committee recommended to Committee 318—Standard Building Code, certain changes in allowable working stresses in bond. These were predicated on the extensive research programs on new types of reinforcing bars which have been in active progress under the sponsorship of the Committee on Reinforced Concrete Research of the American Iron and Steel Institute at various educational and governmental laboratories during the last six years. The data are believed to justify higher working stresses in bond.

The principal revisions recommended were those relating to improved deformed bars which will comply with the proposed revisions of Tentative Specifications for Minimum Requirements for the Deformations of Deformed Steel Bars for Concrete Reinforcement—A.S.T.M. Designation A 305-47 T. The major changes in allowable working stresses in bond for these bars placed in all positions except when used as top bars in beams or girders are as follows:

> Ten percent of the concrete strength for straight bars with no additional allowance for hooks. Tests have demonstrated that the new types of deformed bars supply bond where it is needed—and the addition of hooks adds little or nothing to their effectiveness. It is now well-known that horizontal bars placed in the tops of beams or girders are deficient in bond resistance because of settlement of the concrete and the resulting reduction of surface contact. Therefore, the allowable working stress in bond for the improved types of deformed bars for this situation was recommended to be 7 percent of the concrete strength.
>
> Corresponding values for plain bars hooked at the ends were recommended to be 4½ percent of the concrete strength for the general case, and 3 percent of the concrete strength for bars placed in the tops of beams or girders. No values for straight plain

bars were recommended because it is believed that hooks are needed to supply adequate anchorage wherever such bars are used.

In accordance with current design practice for 2-way footings, allowable working stresses in bond are held to 80 percent of the values used above.

In all cases, ceilings or maximum values are specified as in the present ACI Standard Building Code. The indication is that in the relatively near future any deformed bar not meeting the new A.S.T.M. specifications for the deformations of concrete reinforcing bars will have no standing as a deformed bar and must be used as a plain bar.

Committee 210—Resistance to Erosion in Hydraulic Structures

Lewis H. Tuthill presided in the absence of Chairman Walter H. Price at a meeting attended by three members or their representatives and about ten visitors. The principal discussion centered around the action and nature of cavitation. The point was made that the action of cavitation could not be resisted with concrete and that hydraulic designs and construction would have to be such that low pressure areas causing cavitation would be eliminated. A brief motion picture film combining pictures taken at the Hydraulic Research Laboratories of California Institute of Technology, and the Massachusetts Institute of Technology was exhibited to show in extremely slow motion the action and nature of cavitation. All present were impressed with the tremendous forces involved in cavitation action and expressed appreciation of the opportunity to see this unusually informative film.

Committee 213—Properties of Lightweight Aggregate Concrete

Chairman Washa opened the meeting by summarizing the effort of the Committee to date. He cited the questionnaire sent out to producers and distributors of lightweight aggregate, and stated that the return was about 75 percent with the information supplied ranging from excellent to unsatisfactory. Mr. Washa also cited other sources of information on lightweight aggregates and lightweight-aggregate concretes that have become available in recent months, such as the Bureau of Standards and Bureau of Reclamation reports and a paper by R. E. Davis presented at a recent A.S.T.M. Convention.

He explained that no detailed effort had been made toward a committee report as yet because of the difficulty in assembling complete information and the uncertainty as to best mode for classifying the information. All present agreed that the task of reporting the properties of the various lightweight aggregate concretes in clear-cut fashion was not easy. It was suggested that unit weight of the aggregate and of the concrete serve as a basis for classification. Objections were raised that such a basis might lead to misconceptions and error.

The chairman proposed dividing the Committee's report into 2 or 3 parts. The first would deal exclusively with the aggregates, the second with the concretes made therefrom, and a possible third part dealing with the techniques employed in the proper use of the aggregates for concrete. He also submitted a tabular arrangement of information on some lightweight aggregates which might serve as the pattern for the first section of the report. Each member was asked to study the table critically and at his earliest convenience report to the chairman approval of the table, suggestions for changes or any alternate approach. Attention was called to a number of new aggregates making their appearance on the market. It was pointed out that information and data concerning these aggregates to be included in the report should be supported by authoritative tests by accredited laboratories.

Committee members present were: G. W. Washa, Chairman, C. C. Carlson, Ralph W. Kluge, F. E. Richart, A. G. Timms, L. H. Tuthill and H. F. Williams.

Committee 314—Rigid Frame Bridges

Members in attendance were D. H. Pletta, Chairman, Milton Brumer, and Douglas McHenry. Several non-committee members were present and reported on or discussed research work under way for the committee. The principal project is the test of a one-tenth scale, three dimensional model of a skewed rigid frame bridge. The model is to be made of reinforced concrete with properties similar to the material used in the prototype. This work is being carried on in the Applied Mechanics Dept. at Virginia Polytechnic Inst. by Henry J. Banach, instructor, under the direction of Professors Frederick and Pletta. The prototype of this bridge was completed recently on Long Island, New York. Complete design calculations, based both on the elastic and the plastic theories are available and were the subject of a paper, "Comparative Designs of a Segmented Skewed Frame Concrete Bridge by the Straight Line and Plastic Theory Methods," by Milton Brumer published in the January 1949 ACI JOURNAL.

The discussion centered mainly around the best means of making the test, especially of measuring the variation in the reactions along the hinged base. Present plans call for measuring strains with resistance gages in both longitudinal and transverse reinforcing in the deck, due to concentrated loads placed on the deck, so as to construct influence surfaces. Simultaneous deflections of the structure and foundation reactions will also be measured. It may also be possible to include a measurement of strain due to cyclic temperature changes, by accelerating these variations with a change in the frequency of this thermal condition. The model will then be loaded to failure to check the application of the plastic theory to this type of structure.

The committee wishes to thank R. E. Glover, Frank Baron, H. J. Banach, D. Frederick and H. Simpson for their suggestions and participation in the discussions.

Committee 315—Detailing Reinforced Concrete Structures

Under the Chairmanship of Raymond C. Reese Committee 315 met on Thursday evening. The committee spent hours in making a detailed check of all recommendations received from ACI Members and users of the manual, and compiled by Chairman Reese for consideration as possible changes in the manual. In most cases, the changes are considered minor and are chiefly for clarity and to bring about consistency between sections of the Detailing Manual and the ACI Building Code. Nine members and visitors were present.

Committee 318—Standard Building Code

Fourteen members of the committee and numerous visitors were present at a meeting saddened by the recent deaths of Chairman A. J. Boase and Member T. D. Mylrea. In accordance with a request of Chairman Boase before his death Chester L. Post presided at the meeting.

The committee considered the report of Sub-Committee 6 covering recommended revisions to Sections 701, 702, 704, 708 and 709 of the "Code." These revisions briefly were as follows: 701—a modification and expansion of the empirical moment coefficients; 702—a method for determining moments of inertia I of structural members for purposes of computing their relative stiffness; 704—more definite requirements for beams having laterally unsupported portions; 708—expansion of the section to include two-way concrete joist construction; 709—the inclusion of a definition for "securely attached" as applied to two-way slabs. The recommended revisions to Section 704 were rejected, but all other revisions were approved substantially as proposed.

Professor Richart presented the report of Committee 208—Bond Stress, which report will be referred to Sub-Committees 4 and 7 for study and recommendations regarding inclusion in the "Code."

A summary of the report of Committee 208 appears on p. 14.

FIRSTS for PRESTRESSING and PLASTIMENT

Plastiment helped make the first prestressed concrete floor a successful installation (request reprint Eng. News Record, Jan. 18, 1949 issue).
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Committee 321—Design of Reinforced Concrete Slabs

Under the Chairmanship of N. M. Newmark, Committee 321 met and discussed work of the committee as it might be affected by the Siess-Newmark paper presented at the convention.

Committee 323—Prestressed Reinforced Concrete

Members present were A. E. Cummings, Chairman, Herman Schorer and Ivan Rosov. Visitors were L. Coff, J. J. Creskoff, Curzon Dobell, R. E. Glover, F. R. McMillan, A. S. Neiman, N. M. Newmark, Robert Sheridan, E. Schmid and J. A. Vila. H. B. Zackrison, a member of the committee, was unable to attend the meeting because of other business, and Douglas Parsons, also a committee member, was unable to attend because of ill health.

One of the first questions discussed was the subject of additional committee members. Niels M. Plum, Director of the Danish Institute for Building Research, was invited by action of the Technical Activities Committee to become a committee member and accepted several days before the meeting. Professor Magnel of Belgium has also been invited to become a member of the committee, but he has not yet accepted.

It was agreed that Messrs. Dobell, McMillan and Newmark be recommended to the Technical Activities Committee of the Institute for membership.

After some discussion, it was agreed that there should be some sort of liasion between Committee 324—Precast Reinforced Concrete Structures, and Committee 323—Prestressed Reinforced Concrete. A recommendation was made to TAC that A. Amirikian, Chairman of 324, should be made a member of Committee 323 and that A. E. Cummings, Chairman of 323, should become a member of Committee 324.

Mr. Dobell reported on the organization of an International Committee on Prestressed Concrete which is being set up in Europe. It was agreed that some method of cooperation between the activities of the European committee and the work of Committee 323 should be arranged. The statement was made that the American Water Works Association is preparing specifications for prestressed pipe and tanks, and the desirability for cooperation between the ACI committee and the A.W.W.A. was also discussed.

An adopted working plan provided that sub-committees of two men each should be appointed to carry on various phases of the committee work. The sub-committees proposed are as follows: Sub-committee I—Liaison with European Committee, ACI Committee 324, A.W.W.A., etc., Sub-committee II—Bibliography, Sub-committee III—Patents, Sub-committee IV—Specifications, Notation and Definitions and Sub-committee V—Design and Research.

Subject of many patents, both in the United States and abroad, some engineers hesitate to use prestressed concrete because of the possibility of patent suits. This is particularly true of engineers in public bodies. Mr. Dobell has collected a considerable amount of information on the patent situation and the work of Sub-Committee III will include the preparation of a list of patents and a summary of the patent situation as it stands.

In the matter of a bibliography, it was found that several bibliographies were already available. There is a large bibliography in Professor Magnel's book and the Engineering Societies Library has just published a bibliography on prestressed concrete based on a search of the library files. Other bibliographies are available, including a partial bibliography collected by A. J. Boase when he was chairman of the committee. After some discussion, it was agreed that a new bibliography would be prepared by Subcommittee II on the basis of those already available. It was agreed also that the bibliography would be divided into three sections, (a) Design and Theoretical Calculations, (b) Testing and Experimental Work and (c) Completed Jobs.

As soon as arrangements have been completed for enlarging the membership of the committee, the chairman will make sub-committee assignments so that the work can begin.

ACI Committee 324—Reinforced Precast Concrete Structures

The meeting was attended by the chairman and a majority of the committee members (Messrs. Anderson, Billner, Corbetta, Mitchell and Post), as well as by many guests who had expressed special interest in the committee's work. In his introduction, the chairman briefly described the scope of the committee's work, its activities to date and submitted for consideration and discussion a preliminary outline for a proposed code on precast concrete construction. The ensuing discussion centered primarily on special requirements dealing with minimum cover, bar spacing, concrete quality, permeability, casting inspection and other items which may deviate from the standard ACI Code and which, after approval by the Institute, will serve as a basis in securing building permits for precast concrete structures. A brief progress report of a research project sponsored by the committee at Va. Polytechnic Inst. dealing with corrosion of reinforcing steel in thin precast sections, was also presented. Members were requested to make further studies of the proposed outline and submit comments and suggestions for incorporation in the elaborated draft of the code. It is planned to have the code ready for submission at the next ACI convention.

Committee 613—Recommended Practice for the Design of Concrete Mixes

A very lively meeting was held with 10 committee members or their representatives and 17 guests present. Three subcommittees were appointed to investigate the following points: 1) what percentages of air should be recommended for air-entrained concrete? 2) should the committee report include an alternate method based on the dry, rodded volume of coarse aggregate in a unit volume of concrete? and 3) should information be given in the report showing the effect of slump, richness of mix, grading of aggregate, type of agent, temperature, etc. on the amount of air entrained for a given amount of agent?

It was decided to retain Table 1 of the present committee report which lists recommended water-cement ratios. It was agreed that the strengths shown in Table 2 should be reduced by 5 percent for each percentage of air entrained, which could be covered by a note, and that the statements regarding the slump of concrete should not be changed except that a note should be added that mass concrete should always be vibrated.

Committee 621—Aggregate Selection, Preparation and Use

The following committee members were present, in person or by proxy: I. L. Tyler, Chairman, A. T. Goldbeck, A. A. Levison, R. W. Spencer, A. G. Timms, Stanton Walker and H. T. Williams (representing Fred Hubbard). The meeting opened with a discussion of the scope of the committee's activities as outlined in the assigned task. Mr. Walker asked if that portion of the assignment under "use" could not be dropped. There was general agreement that the use of aggregates has been covered by reports of other ACI committees and that this topic should form a minor part of the Committee 621 report.

The matter of "preparation" received considerable attention. Opinions were divided as to the degree of coverage which should be given the subject.

It was unanimously agreed that the subject of "selection" should be the first considered. A suggestion was made that L. H. Tuthill be asked to summarize the Rhoades-Mielenz paper for use as a basis for selection of aggregates. Mr. Walker agreed to prepare a section on aggregate grading. Mr. Hubbard is being asked to contribute his comments on slag aggregate. Mr. Goldbeck agreed to discuss particle shape and texture of crushed aggregates. The chairman is to attempt an outline for the committee's consideration as a guide to the form of the report.

Committee 711—Precast Floor Systems for Houses

At two sessions the committee discussed the topic of concrete floor joists which, as now manufactured, fail to meet the requirements of "Minimum Standard Requirements for Precast Concrete Floor Units" (ACI 711-46) and "Building Code Requirements for Reinforced Concrete (ACI 318-47). There seemed to be agreement that some variation from specified requirements might be justified but present available data are not sufficient to justify recommending a quantitive change. It was suggested, and one member agreed to look into the possibility of having a manufacturer donate a series of joists for test by committee members at different universities. Information desired and test procedures were given preliminary discussion.

Further discussion pointed to the interdependence of ACI committees. For example: the problem of Committee 711 for spacing reinforcement in joists is tied in with consideration of bond stress, the subject of Committee 208's assignment. Soon will come the use of prestressing by the precast joist manufacturers which should tie in the work of 711 with that of Committee 323—Prestressing Reinforced Concrete. Out of this discussion came the suggestion that the several committees hold a joint session at the next convention to develop an understanding of the problems which are faced by each committee.

Members present were: F. N. Menefee, Chairman; R. E. Copeland; C. F. Moore; Dean Peabody; Gayle B. Price and C. A. Willson. Visitors were: F. C. Bamman; Jack E. Counts; A. C. Grafflin; E. R. Hicks; Dudley Newton and E. R. Smallhorn.

Prize competition for Spanish housing development announced

El Instituto Técnico de la Construcción (Technical Institute of Construction), Madrid, Spain, has announced a prize of one hundred thousand pesetas (about $9000) in an international competition for the best project of industrial development necessary to manufacture machinery, accessories and materials to build dwellings to house 50,000 Spanish families each year. The competition was conceived by this non-profit organization to alleviate the critical housing shortage. Full details regarding the competition may be obtained from the Institute's headquarters, Calle Ruiz de Alarcón, 25, Madrid, Spain.

Who's Who This Month

Robert F. Blanks

Mr. Blanks appears, not in his usual role of author of a technical paper, but as retiring president of ACI in depicting "Concrete of the Future," p. 565 in which is recorded his remarks at the 45th annual convention.

Stanton Walker

Past-president Walker is well known to ACI members both administratively and technically. His work with air entrainment and concrete durability is reported in several JOURNAL papers.

From his experience as Director of Engineering, National Sand and Gravel Assn. and National Ready-Mixed Concrete Assn. he writes of "Controlling the Uniformity of Ready-Mixed Concrete," p. 569.

Walter H. Price and William A. Cordon

Messrs. Price and Cordon give Bureau of Reclamation findings and recommendations in "Tests of Lightweight Aggregate Concrete Designed for Monolithic Construction," p. 581.

Mr. Price is head of the Materials Laboratories of the U. S. Bureau of Reclamation, Denver, Colo. He has been employed by the Bureau since 1930; the past 15 years in materials testing and research related to the construction of large concrete structures. Prior experience includes structural design of Boulder Dam spillways and power plant and hydraulic model studies in connection with spillway designs. Previous ACI papers are "Properties of Concrete Mixes," April 1940, and "Erosion of Concrete by Cavitation and Solids in Flowing Water," May 1947.

William A. Cordon is materials engineer in the research laboratories of the Bureau of Reclamation with previous work as concrete engineer and chief earthwork inspector on several projects, and construction inspector at Grand Coulee Dam, Wash. He was co-author of "Practices, Experiences and Tests with Air-entraining Agents in Making Durable Concrete," published in the Feb. 1949, JOURNAL and delivered at the 45th annual convention by Robert F. Blanks. Mr. Cordon's paper "Entrained Air—A Factor in the Design of Concrete Mixes," appeared in the ACI JOURNAL, June 1946.

J. F. Barbee

Present state of knowledge on air-entraining concrete is reviewed by J. F. Barbee in "What Have We Learned About Air-Entraining Concrete?", p. 601. Since 1932 he has been engineer in charge of the concrete section, Ohio State Highway Testing and Research Laboratory with a principal interest in construction of durable concrete pavement. He received his BS in CE from Ohio State University in 1930. He is a member of A.S.C.E. and N.S.P.E. and ACI.

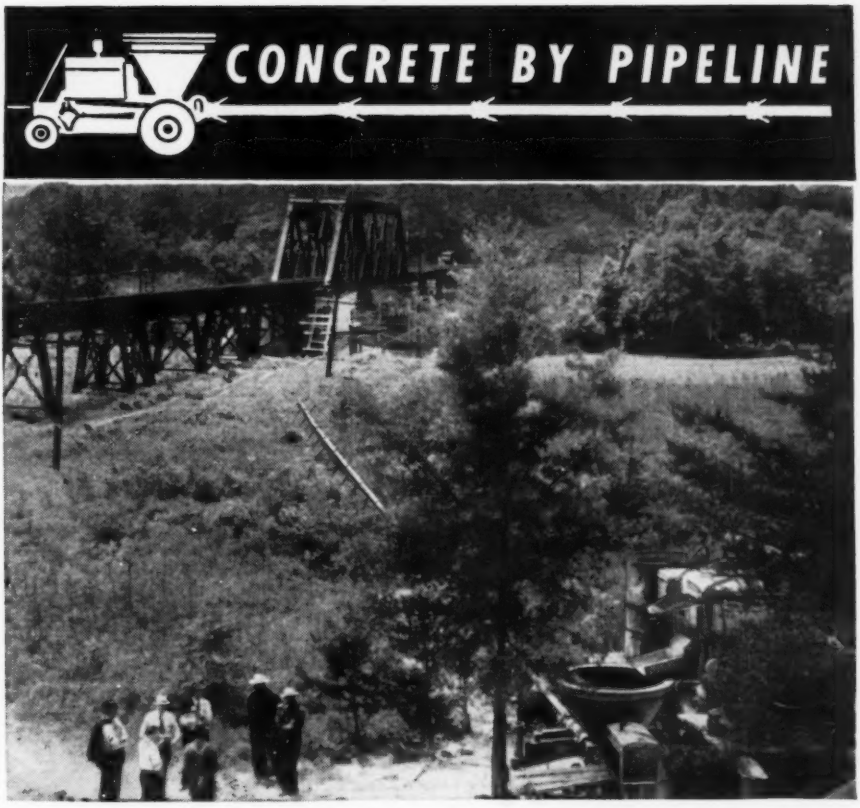
Jay C. Ehle

Production manager of the Cleveland Builders Supply Co., Mr. Ehle describes new devices and techniques of the concrete products industry in "Developments in the Manufacture and Technique of Concrete Masonry Units," p. 613.

With the Cleveland concern since 1938, Mr. Ehle has worked with ready-mixed concrete products and with masonry building units. For one year, 1945, he was manager of the heat treat department of the Stearns Manufacturing Co., Adrian, Michigan.

He is a director and chairman of the Technical Committee of the National Concrete Masonry Assn.

A 1938 graduate of Kenyon College, Mr. Ehle later attended Fenn College specializing in civil and mechanical engineering.



Quality with Economy

Rex Pumpcrete, the pump that pumps concrete through a pipeline, assures highest quality concrete because it reconditions the mix . . . transports it in a sealed pipeline, thus eliminating segregation. The concrete arrives at the end of the pipeline in the same high-quality condition as when it left the reconditioning hopper. "Concrete by pipeline" provides an efficient, economical method of concrete placement.

For example, on this job the Pumpcrete was used to place concrete around the old masonry of two main railroad bridge piers. The Pumpcrete was installed on an existing spur track *450 feet* from the construction site, and the concrete pumped to the piers. It's typical of the ability of Pumpcrete to place concrete without interfering with traffic or other operations.

On this job, as on so many others where quality concrete and fast, economical placement are required, the engineers recommended the Pumpcrete method. For facts on the Pumpcrete method, write Chain Belt Company, 1713 W. Bruce St., Milwaukee 4, Wis.

CHAIN BELT REX COMPANY CONSTRUCTION MACHINERY

New Members

Of the 126 applications approved for membership for the month of February there were 61 Individual, 5 Corporation, 7 Junior and 53 Students.

Illinois was our top membership state for the month providing 39 new Members. (Member getters might seek expert advice from Prof. Newlin D. Morgan, Univ. of Illinois.) New York was second with 12 applicants.

Our membership total February 28, 1949 became 4585.

Alabama

Faircloth, J. M., (Indiv.) University of Alabama, University, Ala.

Arizona

Van Wormer, Bernard T., (St.) 2118 W. Washington St. No. 4, Phoenix, Ariz.

California

Chu, Kao-Shan, (St.) 1623 Shattuck, Berkeley 9, Calif.

Gearheard, Roger D., (Indiv.) 1533 Ridgeway Dr., Glendale 2, Calif.

Pope, Harold F., (Indiv.) 6846 Pacific View Dr., Los Angeles 28, Calif.

Colorado

Peterson, Albert L., (Indiv.) P. O. Box 1499, Denver 1, Colo.

Tyler, Curtis L., (Indiv.) R. 2, Box 422, Arvada, Colo.

District of Columbia

Jain, Jainti Prasada, (Indiv.) c/o Indian Embassy, 2107 Massachusetts Ave., N. W., Washington, D. C.

Florida

Gibbons, Ashby Trice Jr., (St.) 349 Blanca Ave., Tampa, Fla.

Illinois

Bell, Richard Russell, (St.) 340 Whitelaw Ave., Wood River, Ill.

Bianchini, Albert C., (St.) 1116 W. California, Urbana, Ill.

Brejcha, Raymond, (St.) 3308 W. 61st St., Chicago 29, Ill.

Christiansen, John V., (St.) 802 Woodbine Ave., Oak Park, Ill.

Corning, Leo H., (Indiv.) 1524 Fargo Ave., Chicago 26, Ill.

Daniel, James R., (St.) 709 W. Hill St., Champaign, Ill.

Deeming, William B., (St.) 241-2 Court B, Parade Ground Unit, Champaign, Ill.

Dickerson, Ray C., (Indiv.) No. 3 Flora Court, Champaign, Ill.

Doyle, Richard Lee, (St.) 1600 Carolina Dr., Urbana, Ill.

Dulgeroff, Sylvester A., (St.) 512 E. Park St., Champaign, Ill.

Ekstrom, Clarence Theodore, (St.) 10238 Calumet Ave., Chicago 28, Ill.

Evans, Carlisle E., (St.) 1326 42nd St., Rock Island, Ill.

Fina, John Jay, (St.) 507 S. Randolph, Champaign, Ill.

Forsgren, John Delbert, (St.) Ct. F., Bldg. 39, Apt. B-1, Stadium Terrace, Champaign, Ill.

Foulk, Warren Hillon, (St.) 1005 Second St., Champaign, Ill.

Gramley, William E. Jr., (St.) 1304 W. Stoughton St., Urbana, Ill.

Hanner, Verdell, (St.) Route No. 1, East Alton, Ill.

Hapke, Richard Leo, (St.) 807 E. Gift, Peoria 4, Ill.

Havlik, George Millard, (St.) 616 E. Daniel St., Champaign, Ill.

Jobusch, Wallace Emig, (St.) 713 W. Harvard, Champaign, Ill.

Johnson, Charles Norman, (St.) 321-1 Court E, Parade Ground Unit, Champaign, Ill.

Kemper, Calvin C., (St.) 5102 W. Huron St., Chicago 44, Ill.

Lahey, William Stephen Jr., (St.) 604 E. Armory, Champaign, Ill.

Llewellyn, Joseph P., (St.) 410 Blackstone Ave., La Grange, Ill.

Makos, Edmund S., (St.) 1965 S. Trumbull Ave., Chicago 23, Ill.

McGovney, Chester Arthur, (St.) 633 S. 20th Ave., Maywood, Ill.

Michalos, James P., (Indiv.) Technological Institute, Northwestern University, Evanston, Ill.

Nitto, Peter J., (St.) 131 Central Park Ave., Wilmette, Ill.

Poust, Grover O., (St.) 709 Robinson St., Danville, Ill.

Rea, Robert F., (Indiv.) Zonolite Co., 135 S. LaSalle St., Chicago 3, Ill.

Reiter, Melvin Edward, (St.) R. R. 1, Box 17, New Lenox, Ill.

Rubenking, Glen Brochl, (St.) 202 E. Green St., Champaign, Ill.

Rudd, Tony Severe, (St.) 412 E. Park St., Champaign, Ill.

Rydeck, Edwin H., (St.) 10141 Ave. M., Chicago 17, Ill.
Schewe, Edward C., (St.) 1101 Jefferson St., Beardstown, Ill.
Schneider, James F., (St.) 1215 S. Fourth St., Champaign, Ill.
Stansbury, James Edward, (St.) 909 W. Main, Urbana, Ill.
Thorson, George Alfred, (St.), 202 E. Daniel, Champaign, Ill.
Unteed, Kenneth E., (St.) 145 Evanslawn Ave., Aurora, Ill.
Vogelsong, Jack Dale, (St.) 203 E. Green St., Champaign, Ill.

Indiana

Coxen, Forrest Richard, (St.) 942 N. Bancroft St., Indianapolis, Ind.
Erie Materials Co., Inc., (Corp.) 903 Fort Wayne Bank Bldg., Fort Wayne 2, Ind. (Walter J. Fishering)
Seehausen, Richard Ferdinand, (St.) Beanblossom Ridge, Nashville, Ind.

Iowa

Solis-Hegel, Roberto, (St.) Box 120, Friley Hall, Ames, Ia.

Kansas

Middleton, W. B., (Indiv.) 107 N. Steuben, Chanute, Kans.

Louisiana

Barton, John W., (Indiv.) Anderson-Dunham Concrete Co., P. O. Box 1028, Baton Rouge 2, La.

Maryland

Stevenson, Herman Fulmer, (Indiv.) c/o Faisant & Kooken, Consulting Engrs., 2017 Maryland Ave., Baltimore 18, Md.

Massachusetts

Perry, George Nelson, (Indiv.) 105 Greenwood Lane, Waltham 54, Mass.
Yarar, Rifat, (Indiv.) 440 Newbury St., Suite 22, Boston, Mass.

Michigan

Laitsch, Charles E., (Indiv.) 567 Lawndate Ct., Holland, Mich.
Ohman, Warner, (Indiv.) 415 Catalpa Dr., Birmingham, Mich.
Seymour, George, (St.) 502 Rodd St., Midland, Mich.

Missouri

Andrews, William Allen, (Jr.) 5646A Chamberlain, St. Louis 12, Mo.
Beatty, Fred, (Jr.) Box 8181, Plaza Station, Kansas City, Mo.
Campbell, Neal J., (Indiv.) 911 Locust St., St. Louis 1, Mo.

Nebraska

Sherwood, Hugh L., (Indiv.) Box 8, Station C, Lincoln, Nebr.

New Jersey

Campbell, Edward, (Indiv.) East Ave. & Walnut Rd., Vineland, N. J.
Landwehr, Theron, (Indiv.) No. 28 Cottage Place, Keansburg, N. J.
Sollenberger, Norman J., (Indiv.) 146 Hodge Rd., Princeton, N. J.
Voshinin, Igor V., (Indiv.) 2 Bergen Ct., Apt. 2C, Bayonne, N. J.

New Mexico

Wood, Earl P., (Jr.) 480 Washington Ave., Santa Fe, N. M.

New York

Barter, Leland L., (Indiv.) 83-15 Vietor Ave., Elmhurst, L. I., N. Y.
Cameron, Dugald J., (Indiv.) 138 Bruckner Blvd., New York 54, N. Y.
Duggan, Howard W., (Indiv.) Chief Engineer, Stebbins Engineering & Mfg. Co., Watertown, N. Y.
Ehrenpreis, David B., (Indiv.) 1637 W. 13th St., Brooklyn, N. Y.
Gershtenbleet, Leonard, (Jr.) 630 Riverdale Ave., Brooklyn, N. Y.
Horewicz, Julius, (St.) 500 Riverside Dr., New York 27, N. Y.
Kirkpatrick, Reginald A., (Indiv.) c/o Roberts & Schaefer Co., 116 John St., New York 7, N. Y.
MacLaughlin, Walter S., (Indiv.) 30 Wallace Rd., Binghamton, N. Y.
Pytowski, Witold, (Indiv.) 628 W. 114 St., New York 27, N. Y.
Quirk, William H., (Indiv.) Contractors & Engineers Monthly, 470 Fourth Ave., New York 16, N. Y.
Shimamoto, George G., (Indiv.) c/o Kelly & Gruzen, 220 Broadway, New York 7, N. Y.
Walker, John Stanton, (Indiv.) 13 Nesaquake St., Port Washington, L. I., N. Y.

North Dakota

Ehrenhard, Philip E., (Indiv.) Box 275, Glen Ullin, N. D.

Ohio

Faulwetter, Arthur J., (Indiv.) 6821 Hammerstone Way, Cincinnati 27, Ohio
Nicholson Concrete Co., (Corp.) Box 1644, Toledo, Ohio (J. A. Nicholson)
Stein, Ned R., (St.) 132 Windingway, Dayton 9, Ohio

Oregon

Richardson, Henry Leo, (Indiv.) Corps of Engineers, McNarry Dam, Umatilla, Ore.

Pennsylvania

Britton, Raymond E., (Indiv.) 510 Clen-Moore Blvd., New Castle, Pa.
DeGroot, Archibald, (Indiv.) 1009 Wyoming Ave., Scranton 9, Pa.
Loewer, Alvin C. Jr., (Indiv.) 1861 Snyder St., Bethlehem, Pa.

South Carolina

McKoy-Helgerson Co., Inc., (Corp.) 8-A. W. North St., Box 25, Greenville, S. C. (R. D. Frey)

South Dakota

Schuneman, Dean I., (St.) Barracks 199, Brookings, S. D.

Tennessee

Holman, Joseph W., (Indiv.) 702 Stahlman Bldg., Nashville, Tenn.
Watson, Alpheus Clifton, (St.) 18 Young St., Bemis, Tenn.

Texas

Oehler, LeRoy Theodore, (Indiv.) 113 E. D. Hall, Civil Engineering Dept., University of Texas, Austin, Texas

Washington

Jackson, Alfred H., (Indiv.) P. O. Box 573, Richland, Wash.
Means, Robert R., (Indiv.) Alaska District, C/E, APO 942, c/o P. M., Seattle, Wash.

West Virginia

Tow, Marvin H., (Indiv.) Bellepoint, W. Va.

Wisconsin

Dettinger, Glenn W., (St.) 214 N. 6th Ave., Wausau, Wis.
Irgens, Rolf Nelson, (St.) 2424 N. Harding Blvd., Wauwatosa 13, Wis.

Puerto Rico

Ortiz, Humberto, (Indiv.) 404 Bolivar St., Apt. 2, Santurce, Puerto Rico
Rivera, Francisco J. Silvestre, (Indiv.) Box 6432, Loiza St. Station, Santurce, Puerto Rico
Silva, Francisco, (Jr.) 64 Luisa St., Santurce, Puerto Rico

Austria

Drogsler, Otto, (Indiv.) Silbertal 90 Post Schruns, Vorarlberg, Austria
Friedrich, Erich, (Indiv.) Technische University, Rechbauerstrasse 12, Graz, Oesterreich, Austria

Canada

Raud, Juhan, (Indiv.) Hydro-Electric Power Comm., Consulting Engr. Div., 620 University Ave., Toronto 2, Ont., Canada

Colombia

Tejada-Saenz, Jose, (Indiv.) c/o Empresa de Energia Electrica, Medellin, Colombia, S. A.

Costa Rica

Aguilar, Alvaro Robles, (St.) Paseo Sarmiento No. 346, San Jose, Costa Rica, C. A.
Alvarez, Alfonso, (Indiv.) Dpto. de Arquitectura y Construcciones, Obras Publicas, San Jose, Costa Rica

Cuba

Duplace, Luis Saenz, (Indiv.) Soledad 205—Ap. 703, Havana, Cuba
Gutierrez y Lopez del Castillo, Alfonso, (St.) Calle 12 No. 710 e/Ave. 9 y Ave. 10, Ampliacion de Almendares, Marianao, Habana, Cuba
Raventos, Ramon Acosta, (St.) Consulado No. 562 esq. 11, Ampliacion de Almendares, Marianao Havana, Cuba

Egypt

Yehya, E. Mostafa, (Indiv.) 5, Chawky St., Guiza, Cairo, Egypt
El-Erian, Ahmed Ali, (Jr.) 11, Nasralla Fanous St., Giza (Balad) Egypt

England

Champion, Stewart, (Indiv.) 4, St. Martin's Ave., York Rd., Doncaster, Yorkshire, England
James, Michael Llewellyn, (Indiv.) 1 St. Ann's Crescent, London S. W. 18, England
Seville, S. W., (Jr.) 53, Woolley Lane, Hollingworth, Manchester, England

Greece

Moraitis, John N., (Indiv.) Loukianou Ste. 21a, Athens, Greece

Guatemala

Castillo R., Alfonso, (St.) Pequeno Manchen No. 2, Guatemala City, Guatemala, C. A.

Holland

N. V. Betonmortelfabriek Rotterdam "BEFARO," (Corp.) Haringvliet 19, Rottendam Holland (J. A. Bakels)

Honduras

Dominguez Agurcia, Roberto, (Indiv.) Tegucigalpa, D. C., Honduras, C. A.

Continued on p. 32

Positions and Projects — ACI Members

C. L. Post heads Committee 318

Chester L. Post, consulting engineer with the U. S. Public Buildings Administration and long-time ACI member, has been appointed by the Board of Direction as Chairman of ACI Committee 318—Standard Building Code, to succeed Arthur J. Boase. Mr. Post will serve also as ACI representative on the C. R. S. I. award committee of which Frank E. Richart is chairman.

Barnes to consult on Argentine projects

Prof. G. E. Barnes, head of the department of civil engineering and engineering mechanics at Case Institute of Technology, has been appointed consulting engineer to Parsons, Brinckerhoff, Hall and McDonald of New York, associated with Damon, Dinerman & Cia S. A. of Buenos Aires to advise the Argentine government on hydroelectric projects for the Salado and Juramento rivers. Professor Barnes, an ACI member since 1934, was consulting engineer for the U. S. Engineer Dept. on the $40,000,000 Muskingum watershed project.

Couse named A. G. C. vice president

Walter L. Couse of Walter L. Couse & Co., Detroit, was named vice president of the Associated General Contractors of America at the 30th annual convention in New York on March 3, 1949. An ACI member, he serves also on the advisory council of the Senate Interstate Commerce subcommittees on trade policies.

Marsh promoted

Wallace A. Marsh has been appointed general manager of the Permanente Cement Co., Oakland, Calif. He was formerly assistant to Carl R. Olson, who is now vice-president of the company.

Kauer returns to Ohio State Highway Dept.

Theodore J. Kauer has returned to the Ohio State Highway Department as director of highways. Mr. Kauer was employed as a highway engineer by the department from 1927 to 1940, serving as maintenance engineer, construction engineer, and division engineer in Cleveland and in Columbus as federal projects engineer and acting chief engineer of construction. He served for a short time with C. A. A. in Washington, D. C., as an airport paving engineer. In 1941 he became assistant managing director of the American Concrete Pipe Association, where he remained until 1945, resigning to become managing director of the Wire Reinforcement Institute in Washington, D. C.

Morris heads physical laboratory

Lloyd M. Morris, associated since 1924 with the Pennsylvania Railroad, has been named head of the system's Test Department Physical Laboratory with responsibility for all laboratory physical testing. During World War II, Lt. Col. Morris was in charge of the Quartermaster Corps Jeffersonville Laboratory responsible for research, development and testing of equipage, tentage and mechanical equipment.

Moorhead named promotion manager

John W. Moorhead has been made product promotion manager for the Building Products Division, Great Lakes Carbon Corp. of which he is ACI representative. Mr. Moorhead will administer the promotion program for Great Lakes rock wool building insulation products and for Permalite plaster and concrete aggregate.

Mr. Moorhead joined Great Lakes Carbon early in 1947 after 18 years with

the Aluminum Co. of America during which time he was in charge of Alcoa's building industry sales in New York, Chicago and Midwest. During the war he directed housing construction for that company's war time expansion, including Alcoa's Defense Plants Corp. units. He is a graduate in Civil Engineering of Rose Polytechnic Institute, Terra Haute, Ind.

Materials engineer appointed

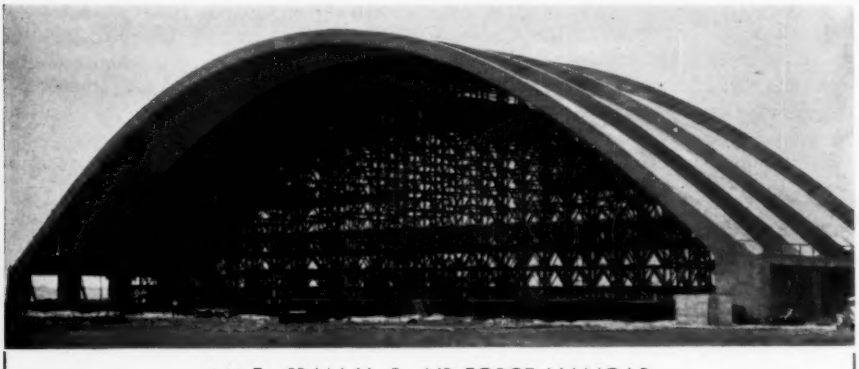
George A. Rahn has been appointed materials engineer, Pennsylvania Turnpike Commission, Harrisburg, Pa. He was previously engineer of materials and construction, Highway Research Board, National Research Council, Washington, D.C.

Ledyard heads PCA committee

Eugene A. Ledyard, Executive Chemist for the Monolith Portland Cement Co. has been appointed chairman of the General Technical Committee of the Portland Cement Assn. This committee is composed of executive, sales, operating and technical representatives of member companies and comprises five working sub-committees. The purpose of the committee is to act in a consulting and advisory capacity to the Research and Development Division of PCA and embraces problems arising with respect to research on cement, its manufacture, its characteristics and performance, methods of testing, ways and means of improving quality and related matters; research on the making of concrete and on the characteristics of concrete as a structural material including studies of methods to improve its quality, uniformity, durability, economy and performance.

Mr. Ledyard has represented the Monolith Portland Cement Co. in PCA matters for the past eight years and last year was chairman of Sub-Committee B on Technical Research of the General Technical Committee.

He is also Monolith representative in the ACI and their representative on Committee C-1 (on Cement) of A.S.T.M. and is a member of the American Chemical Society.



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NEW YORK OFFICE—116 JOHN STREET, NEW YORK 7

MacBride named president of Hercules Cement Corp.

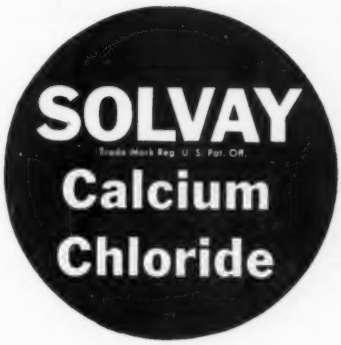
D. S. MacBride, long a member of ACI, has been elected president of the Hercules Cement Corp., Philadelphia, after 12 years with the company as vice president for sales and executive vice president. Prior to joining Hercules, Mr. MacBride had been, first, a district engineer for PCA and later associated with the Lone Star Cement Corp., where as manager of the Incor Div. he played a prominent role in introducing the first high early strength cement in the U. S. Mr. MacBride replaces Joseph Brobston, who becomes chairman of the board of directors. Now in his fiftieth year as a cement manufacturer, Mr. Brobston, ACI member since 1936, was one of the original members of PCA and for more than 30 years has been a member of A.S.T.M. Committee C-1 on Cement.

Corning appointed manager of PCA Structural and Railways Bureau

Leo H. Corning, since 1935 Assistant Manager of the Portland Cement Assn. Structural and Railways Bureau, was appointed Manager of the Bureau April 1, filling the vacancy created by the recent death of Arthur J. Boase.

Mr. Corning graduated in civil engineering and received his B.S. and C.E. degrees from Case Institute of Technology. During World War I, he served as an engineering instructor in the Coast Artillery Corps., U. S. Army. He later practiced as an architect and engineer in Florida, and served as structural engineer with the M. A. Hanna Co. of Cleveland and as engineer of design for the Water Dept. of that city. Immediately before joining PCA, Mr. Corning was structural engineer with the Foundation Co., contractors and engineers of Chicago.

He is a member of ACI serving on ACI Committee 324—Precast Reinforced Concrete Structures, A.S.C.E., Reinforced Concrete Research Council sponsored by the Engineering Foundation, and Sigma Xi.



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James H. Cissel

Prof. James H. Cissel, 59, Univ. of Michigan Engineering School faculty member, died at Ann Arbor, Jan. 29. He had been at the university since 1915 and held a professorship since 1928. While on leave to the Michigan State Highway Department from 1933 to 1936, he designed a bridge to span the Straits of Mackinac. Later he served on the State Bridge Authority. He received a Bachelor of Science degree at Purdue Univ. in 1910, worked as an engineer for five years, and then joined the U. of Michigan staff.

Karl W. Mautner

Karl W. Mautner, civil engineer, died recently in London, England.

Born in Vienna in 1881, he was for many years Professor of Civil Engineering at Aachen Univ. The Nazi regime forced his resignation in 1934, and later he was interned in Buchenwalde Concentration Camp. Upon his release he became a technical advisor to L. G. Mouchel and Partners, Westminster, England.

From 1930 to 1937 he was one of the three technical directors of the Franco-German organization set up to reconstruct Dunkirk Harbor, and he was also a member of the designing board for the highway tunnel under the Maas at Rotterdam.

Civil Engineering Congress in Mexico

An extensive and interesting social and technical program is planned for the First International Congress of Civil Engineering in Mexico to be held at Mexico City April 30 to May 7, 1949. President Aleman will address the congress and cabinet officials will give receptions for engineers and their wives attending the congress. Tours of public works and points of interest have been arranged. Hotel accommodations are to be had at 2-8 dollars per day without meals. Arrangements for travel in Mexico may be made with the nearest Mexican Consulate.

For further information write to the Organizing Committee, Plaza de la Republica 55-606, Mexico, D. F.

Errata—Durability of Concrete Exposed to Sea Water and Alkali Soils —California Experience

An engraving error has been noted in Fig. 7 in the paper by Thomas E. Stanton, "Durability of Concrete Exposed to Sea Water and Alkali Soils—California Experience," ACI JOURNAL, May 1948, *Proc.*, V. 44, p. 821. For those who wish to correct their Journals, Fig. 7 is printed on the opposite page with views of the 4 sack, Type II and 5.5 Sack, Type I specimens with AEA in their correct positions.

A.S.T.M. committee week

Among A.S.T.M. committees which met in Chicago February 28 to March 4, 1949, were C-1 on Cement, C-9 on Concrete and Concrete Aggregates and C-15 on Manufactured Masonry Units.

Comm. C-1, F. H. Jackson, Chairman and G. E. Warren, Secretary, considered the preparation of a specification for air entraining additives for intergrinding with portland cement under Specification C 175. The committee voted to recommend placing a maximum limit of 15 percent on air entrained by cements under Spec. C 150 when tested in standard mortar according to Method C 185. A new Tentative Specification for Natural Cement was prepared to replace Specification C 10.

Comm. C-9, Stanton Walker, Secretary, elected Roy W. Crum and Harry J. Love to honorary committee membership in recognition of their long and valued services. Proposed methods of tests being circulated are concerned with entrained air in freshly mixed concrete, test for soft particles in coarse aggregate, and comparing concretes on the basis of bond developed with reinforcing steel.

All specifications for concrete masonry units were recommended for continuance without change at this time by Comm. C-15. Specification for Drain Tile C 4-24 is under study and proposals for revisions are expected at a future meeting.

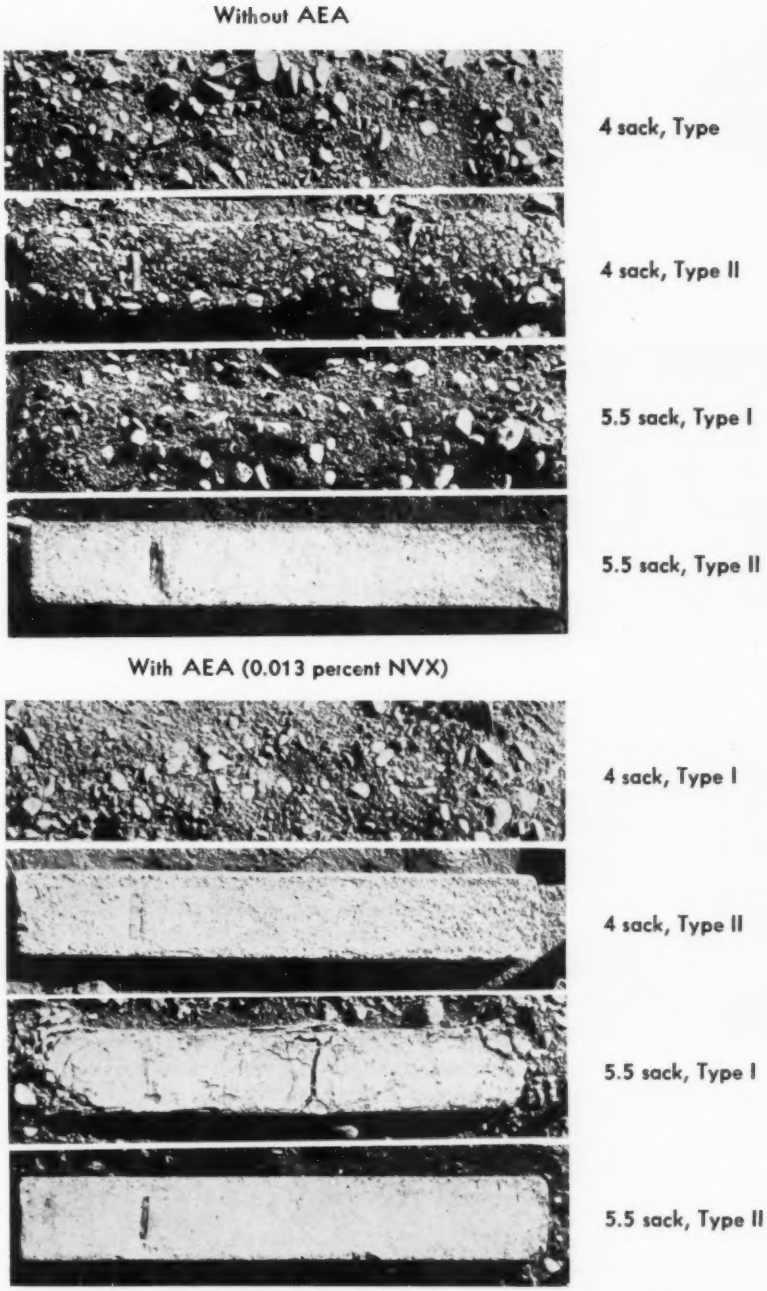
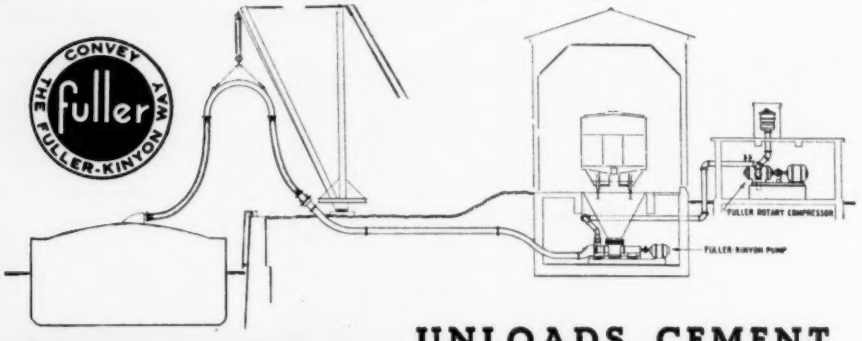
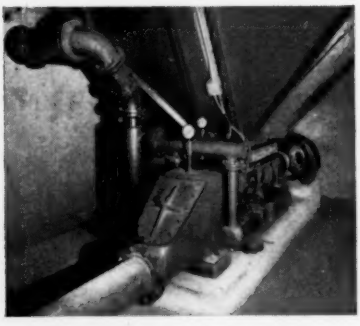


Fig. 7—Concrete with and without an air entraining agent after 30 months exposure to an alkali soil high in Na_2SO_4. The cements were a California Type I (12 percent C_3A) and a Type II (7 percent C_3A) cement of the same brand.

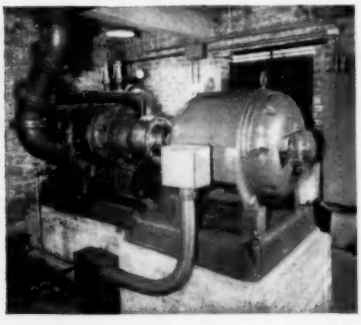
(Correction for paper by Thomas E. Stanton, "Durability of Concrete Exposed to Sea Water and Alkali Soils—California Experience," ACI JOURNAL, May 1948, *Proc.* V. 41, p. 821.)



UNLOADS CEMENT FROM HOPPER CARS . . . CONVEYS TO BARGES



Fuller-Kinyon Pump in pit underneath car



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G-56

Honor Roll

February 1 to February 28, 1949

Our new Honor Roll beginning February 1, 1949 finds last year's Honor Roll top man Prof. Newlin D. Morgan off to a running start with credit for 42½ Members. That's 9½ more than he sponsored in the preceding 12 month period and note that he is professor of architectural—not civil—engineering at Univ. of Illinois.

Jose Antonio Vila is second with 3 credits and William T. Neelands and Leon Venegas tied for third place with 2 credits.

Newlin D. Morgan	**42½**
Jose Antonio Vila	**3**
William T. Neelands	**2**
Leon Venegas	**2**
Paul W. Abeles	1
William F. Anderson	1
Samuel Baker	1
Charles F. Ball	1
John H. Bateman	1
J. F. Baxter	1
Oscar Benedetti	1
Hans H. Bleich	1
R. A. Caughey	1
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Raymond E. Davis	1
Peter J. Doanides	1
W. H. Droege	1
Belmon U. Duvall	1
H. F. Gonnerman	1
J. W. Hubler	1
Emory E. Johnson	1
Barton Jones	1
Karl W. Lemcke	1
Glenway Maxon	1
Carl A. Menzel	1
Jesus Mesa	1
Robert Morris	1
George G. Nichols	1
Raul Miguel Rivero	1
Saul Shaw	1
Thomas C. Shedd	1
Richard Skalak	1
Maurice Taylor	1
Stanton Walker	1
Donald K. Woodin	1

The following credits are, in each instance "50-50" with another member.

C. Harold Barcus	Wm. Morris
Charles H. Barker	Philip M. Noble
Frank Baron	Chester B. Palmer
Roberto Barillas F.	Douglas E. Parsons
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Harold W. Brewer	Jack S. Short
Milton Brumer	Ruben Ruiz Silva
Arthur P. Clark	G. W. Stokes
R. D. Conrow	E. O. Sweetser
Roger H. Corbetta	Anton Tedesko
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J. L. Faisant	Homer Thrall
Phil M. Ferguson	I. L. Tyler
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Albert C. Hooke	Thomas Worcester
Max Krueger	K. W. Wright
Hudson Matlock	Alexander H. Yeates
J. W. McBurney	Robert Zaborowski
T. R. McCollough	

Convention Jabs and Jibes

"Frankly, I was amazed at the slides some of the authors put on the screen. They have been going to conventions long enough to have known better. Some might as well have tried to show the Lord's Prayer engraved on the head of a pin. We shall have to continue harping on these things 'til all authors realize that their cluttered slides are almost worthless. Perhaps we should note, however, that the expense of producing special drawings for slides is sometimes a large consideration and no doubt felt unjustified when the information is already available in existing drawings. But that doesn't help the audience."

"What we need is a clearing house for research work with which research projects might be registered. This would tend to prevent duplication of effort and make possible exchange of information on related topics. As it now is, you know about research in your line only by accident or through informal contacts."

"In my opinion a registration fee ought to be charged at the convention. It is human nature to value more highly that which must be paid for."

New Members

Continued from p. 25

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Vengurlekar, Vithal Ramkrishna, (Indiv.) c/o Hydraulic Engineer's Dept., Bombay Municipality, Bombay, India

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Visitors at ACI headquarters

Recent visitors at ACI headquarters have included 5 engineers who have been traveling and studying in the United States as a delegation from the government of the United Provinces, India. On January 27 Aditya Prakash, O. E. Gupta Yadava Mohan and Mrs. Mohan signed the visitors' register. On March 22 J. P. Jain and R. S. Chaturvedi of the Irrigation Research Station were visitors. All are interested in concrete for dams.

They Registered at 45th Annual ACI Convention

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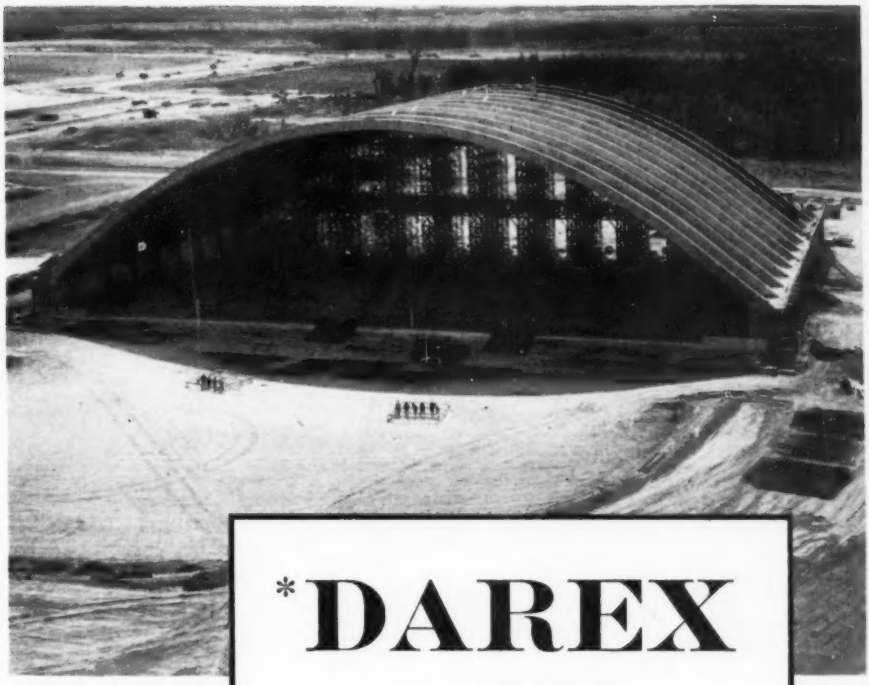

Concrete hangar for B-36's, Limestone, Maine Bomber Base. Courtesy U. S. Corps of Engineers.

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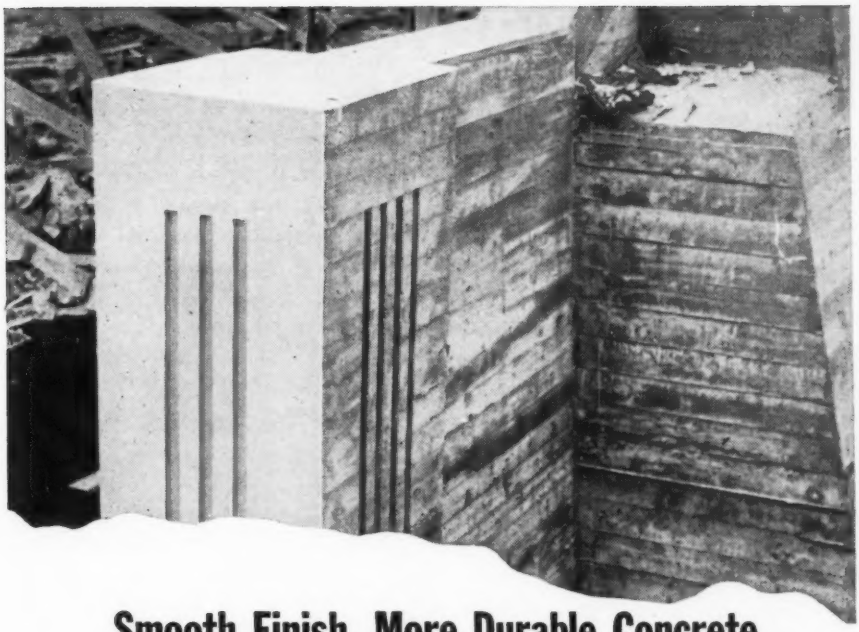

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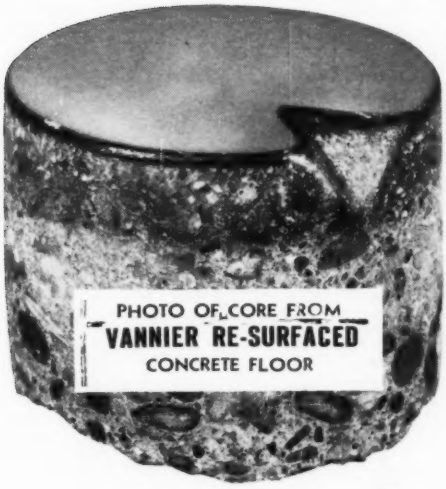

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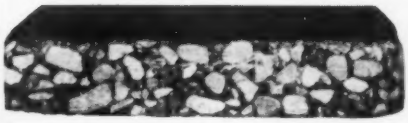

Cross-section of Asphaltic Concrete Wearing Surface showing Aggregate Arrangement Designed to Carry Maximum Loads—both Moving and Static.

The VANNIER METHOD of Re-surfacing Concrete with Concrete

(illustrated above at left)

is practical, not only from the standpoint of DURABILITY but because of the important SAVING IN TIME. The new concrete wearing surface can be completed in from four to six days, depending upon conditions. If our Asphaltic Concrete Surface (illustrated at right) is specified, only twenty-four to forty-eight hours are required.

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Tools, Materials, Services

Under this heading note will be made from time to time of producer literature of presumed technical interest (and available from its source for the asking) to ACI users of tools, equipment, materials, accessories and special services.

New building block

A block claimed to be moisture, fire and frost resistant is being produced for use in building construction. It has an asphalt-treated insulation board built into the center and secured with 16 gage galvanized steel wire.

Further information is available from the Insol Co., St. Paul, Minn.

Rubber-tipped concrete vibrator

Rubber-tipped vibrators have been developed, which the manufacturer claims will reduce damage to expensive forms such as Plaster of Paris inserts, plywood and the popular moisture-absorbent form lining materials such as Celotex.

Further information concerning Viber Vibrators may be obtained from the Viber Co., 726 S. Flower St., Burbank, California.

Concrete drill

A blunt end drill bit is claimed to drill through gravel or rounded rocks in concrete rather than being deflected around them. These drill bits, handmade of high grade steel have heat treated carbide cutters set in and brazed to the blunt end of the bit shaft. Ranging in size from ¼ to 2 in. these drills are said to require infrequent sharpening and to be less subject to breakage.

For further data write Rotary Concrete Drill Co., 650 Arroyo Parkway, Pasadena 2, California

SYNOPSES of recent ACI Papers and Reports

Institute papers of this JOURNAL Volume 20 are currently available at prices indicated. Please order by title and title number.

RECOMMENDED PRACTICE FOR WINTER CONCRETING METHODS (ACI 604-48).....45-1

Price 50 cents.

REPORT of COMMITTEE 604—Sept. 1948, pp. 1-20, (V. 45) in special covers

Supersedes 44-13.

This ACI standard establishes methods of cold-weather concreting for thin sections and mass concrete. Heating of materials, accelerators and anti-freezes, curing and temperature records during curing, subgrade (or base) preparation, protective coverings during curing, and form removal are discussed for both types of job, and preferred methods are indicated. An appendix entry outlines objectives of the special winter methods with background material which indicates the "why" of some of the recommended practices. Charts in the appendix indicate effect of curing temperature on concrete strength, and a list of 135 selected references to periodical literature on winter concreting methods is included.

RELATIVE STRENGTHS OF PORTLAND CEMENT MORTAR IN BENDING UNDER VARIOUS LOADING CONDITIONS.....45-2

Price 35 cents.

JACOB PORTER FRANKEL—Sept. 1948, pp. 21-32 (V. 45)

Assuming the behavior of standard mortar to be similar to that of plain concrete, tests were performed on 99 small mortar beams under sixth-, third- and center-point loading to verify the applicability of the statistical theory of the strength of brittle materials to concrete structures. The theory, as first developed by Weibull, is briefly analyzed, compared to the methods used by Tucker, and finally applied to the tests at hand. Agreement between the theoretical and experimental findings is so close as to warrant future testing on plain concrete specimens.

TERRAZZO AS AFFECTED BY CLEANING MATERIALS.....45-3

Price 35 cents.

D. W. KESSLER—Sept. 1948, pp. 33-40 (V. 45)

Effect of cleaning materials on terrazzo was studied using solutions of the following detergents: soda ash, trisodium phosphate and synthetic sulfonate. Seventy-six different aggregates were used in preparing terrazzo disks which were moist cured 3 months and then surface ground to form a shallow dish. Detergent was allowed to stand in dish 30 minutes before rinsing and drying (at 105 C). This testing cycle was repeated until deterioration tendencies were established. Soda ash solution proved much more destructive than the trisodium phosphate; however, up to the time of writing (after 850 cycles) the synthetic sulfonate had caused no failure.

TRANSPORTING READY-MIXED CONCRETE IN OPEN DUMP TRUCKS.....45-4

Price 35 cents.

R. A. BURMEISTER—Sept. 1948, pp. 41-56 (V. 45).

Milwaukee experience in hauling air-entraining concrete in open dump truck bodies is recorded. Slump, air content, specific weight, compressive strength and workability tests were made on concrete before and after a trial run over bumpy pavement. Some similar tests were made on concrete being placed in a Milwaukee street after open truck transportation. The author concludes that open trucks for hauling ready-mixed air-entraining concrete are satisfactory when air content is between 3 and 6 percent (5 to 6 sacks cement per cu yd) and hauling time is 45 minutes or less, provided that aggregate moisture, grading and slump are carefully controlled.

A STUDY OF ALKALI-AGGREGATE REACTIVITY BY MEANS OF MORTAR BAR EXPANSIONS.....45-5

Price 35 cents.

T. M. KELLY, L. SCHUMAN and F. B. HORNIBROOK—Sept. 1948, pp. 57—80 (V. 45)

Many types of aggregate were combined in varying amounts and sizes with high- and low-alkali cements and formed into 1 x 1 x 10-in. mortar bars. The bars were stored either at 70 F or at 100 F and their expansions measured at ages ranging from 1 month to 4 years.

In combination with high-alkali cements, opal, opaline chert and a siliceous dolomitic limestone were found to cause greatest expansion. Certain aggregates containing volcanic glasses and some natural sands and gravel also caused excessive expansion; with one exception, these sands contained small amounts of opal.

Greatly delayed expansion resulted with the very fine sizes of opal, particularly in combination with high-soda cement. Similar behavior resulted with minus No. 81 size opal and low-alkali cement with either Na_2SO_4 or K_2SO_4 additions.

Materials such as dehydrated kaolin, soda feldspar, magnesium fluosilicate, acetic acid and calcium hydroxide added in small amounts as correctives were ineffective. However, diatomaceous earth in sufficient quantity as a cement replacement eliminated expansion.

REINFORCED CONCRETE WALL AND COLUMN FOOTINGS PART I.....45-6a

Price 60 cents.

FRANK E. RICHART—Oct. 1948, pp. 97-128 (V. 45)

PART II.....45-6b

Price 35 cents.

FRANK E. RICHART—Nov. 1948, pp. 237-260 (V. 45)

This paper presents, in two parts, a report on an experimental investigation of reinforced concrete footings. It represents the first extensive study of the subject since the well-known work of Talbot in 1913. In the meantime there have been many developments in materials, as well as in design methods.

Important developments from these tests include the following: (a) Definite proof that the tensile and bond resistance depend upon the bending moment and shear found by statics by consideration of the full applied load, and not 85 percent thereof, as assumed in current building codes; (b) Hooked ends of bars showed no particular advantage in bond resistance over straight bars, particularly when deformed bars of new improved types were used; (c) Hooked bars produced very little effect on resistance to diagonal tension, much less than is commonly assumed; (d) Welded mats proved particularly effective in resisting end slip of bars; (e) Footing caps or piers are effective in reducing the amount of reinforcement and in increasing the load capacity of footings; and (f) Diagonal tension seems to be the point of weakness in current design practice. The factor of safety of thin footings in this respect appears greater than in thick footings, and is generally greater in rectangular than in square footings when the conventional methods of computation are used.

In these tests, major emphasis has been placed on the isolated column footing. Principal attention has been

given to the resistance of footings to failure by bond, diagonal tension and tension in the steel. Test footings were designed to produce these various types of failure. In addition, studies were made of the behavior of rectangular footings and footings having intermediate capitals or piers.

The tests featured measurements of tensile and compressive strains, deflections and slippage of bars. In some of the tests three grades of concrete were used, in others, five types of reinforcing bar were employed. In most of the tests, the footings were 7 ft square. They were supported on a bed of steel springs and tested in a large testing machine.

This paper merely attempts to present and interpret the test results, and does not offer recommendations as to possible changes in design procedures at this time.

LATTICE ANALOGY IN CONCRETE DESIGN.....45-7

Price 35 cents.

DOUGLAS McHENRY—Oct. 1948, pp. 129-140 (V. 45)

The lattice analogy is a scheme for solving two-dimensional stress problems in which the involved mathematical methods of the theory of elasticity are replaced by simple computations. The solid section is replaced by an equivalent lattice or framework which may be solved by methods applicable to indeterminate structures. Stress distribution in sections of complex shape and with complex loading may be determined by successive approximations which involve only substitution in simple equations. The method is illustrated by application to the problem of stresses in a deep beam with off-center loading.

STRENGTH OF PRECAST CONCRETE FLOOR JOISTS.....45-8

Priced 35 cents.

JACOB FELD—Oct. 1948, pp. 141-148 (V. 45)

The strength and usability of precast concrete floor joists left in the open without any protection for a year, when a housing project was abandoned, were proved by load test on a slab section incorporating the poorest joists. This paper describes the condition of the joists, the load test results and indicates that a greater tolerance can be safely permitted in the visible defects of precast concrete joists.

EFFECT OF ENTRAINED AIR ON CONCRETES MADE WITH SO-CALLED "SAND-GRAVEL" AGGREGATES.....45-9

Price 35 cents.

PAUL KLIEGER—Oct. 1948, pp. 149-164 (V. 45)

Sand-gravel aggregates (maximum particle size ⅜ in.) used with non-air-entraining cement produce concretes containing from 3 to 4 percent air. The same mixes made with air-entraining cement and the sand-gravel aggregates have air contents from 8 to 13 percent. Effect of this larger amount of entrained air is given in terms of tests of flexural and compressive strength, freezing and thawing resistance, and length changes in varying storage conditions. Effect of "sweetening" the sand-gravel by addition of 30 percent of coarser aggregate is noted.

BURNED SHALE AND EXPANDED SLAG CONCRETES WITH AND WITHOUT AIR-ENTRAINING ADMIXTURE.....45-10

Price 35 cents.

P. H. PETERSEN—Oct. 1948, pp. 165-176 (V. 45)

The physical properties of several lightweight aggregate portland cement concretes made with burned shale or expanded slag were investigated at the National Bureau of Standards. Three grades of concrete were made with each aggregate. Air-entrainment greater than 20 percent is reported for the mixtures leanest in cement, an air-entraining admixture being used to increase the workability of all but the richest concretes. Compressive, flexural and bond strength data are given as well as resistance to heat transfer, rain penetration and water penetration by capillarity. Also included are the coefficients of thermal expansion, shrinkage, and values for change in length due to wetting and drying.

CEMENSTONE PRECAST CONSTRUCTION.....45-11

Price 35 cents.

A. C. GRAFFLIN—Nov. 1948, pp. 193-204 (V. 45)

Standardized precast reinforced concrete structural members and panels point toward economy in fireproof construction of ordinary buildings up to four stories. The method described has been so developed that architects and engineers can select from tables reinforced concrete beams, columns, roof, floor, and wall panels in the same manner as for structural steel. Design is in accordance with the ACI Code and costs compare favorably with similar structures framed of steel and fireproofed with concrete.

SAND GRADING INFLUENCE ON AIR ENTRAINMENT IN CONCRETE.45-12

Price 35 cents.

M. A. CRAVEN—Nov. 1948, pp. 205-216 (V. 45)

Four series of mixes with varying air-entraining agents, cement factors, and sand grading and content were prepared in order to observe sand grading effect on concrete. Graphic record is presented of air content and W/C plotted against fineness modulus of sand; flow and compressive strength plotted against fineness modulus of sand; and air content plotted against percent of No. 30—No. 50 sand and total sand percentage. Generally the percentage of air entrained in concrete increased with decrease in fineness modulus of sand. Quantity of air appears to be a function of the quantity of No. 30—No. 50 sand. Effect of grading and quantity of sand on other properties of fresh and hardened concrete is noted.

INFLUENCE OF SIZE GRADING OF SAND ON AIR ENTRAINMENT.45-13

Price 35 cents.

E. W. SCRIPTURE, Jr., F. B. HORNIBROOK, and D. E. BRYANT—Nov. 1948, pp. 217-228 (V. 45)

Field reports indicated difficulty with certain sands in securing the desired amount of entrained air; this was frequently attributed to the size grading of the sand, particularly to a deficiency in the finer fractions. An experimental investigation was undertaken in view of the paucity of published data on this subject. Mixes of sand and water alone, 1:4 and 1:2 mortars, and concrete mixes were made with and without air-entraining agents and the air contents determined. While size grading of the sand had a very large influence on air entrainment in a mixture of sand and water alone, this effect was smaller in mortars and very small in concrete mixes. It was found that maximum air was entrained by the 28-48 mesh size sand rather than the 48-100 mesh size.

USE OF PREPACKED AGGREGATE CONCRETE IN MAJOR DAM CONSTRUCTION.....45-14

Price 35 cents.

PAUL BAUMANN—Nov. 1948, pp. 229-236 (V. 45)

This paper sets forth the method of construction of a major gravity dam whereby the average temperature of the mass concrete may be controlled within the mean annual and the dry shrinkage reduced to a fraction of that of conventional concrete by means of prepacking of the coarse aggregate to a density corresponding to 30 percent voids; precooling the aggregate; the intrusion of the coarse aggregate by mortar with a cement content of two sacks per cu yd or less; and the internal post-cooling, curing and draining of the mass concrete. (This process is known as the Prepakt method of construction.)

RATIONAL ANALYSIS AND DESIGN OF TWO-WAY CONCRETE SLABS.....45-15

Price 60 cents.

C. P. SIESS and N. M. NEWMARK—Dec. 1948, pp. 272-316 (V. 45)

A new method for the design of two-way building slabs is proposed and its step-by-step development is described. A new moment distribution procedure is used to compute moments in a number of rectangular slabs continuous over rigid beams. Several variables are studied in the analyses and include: the ratio of sides, the effect of discontinuous edges, the torsional stiffness of the beam, various types of loading, and combinations of panels of various sizes and shapes. Certain conclusions are stated regarding the types of loading to be considered and the values of beam torsional stiffness to be assumed in the development of the design procedure.

Moments obtained in the foregoing analyses are modified to take into account the effect of additional variables. First, the slab moments are increased by various amounts as a result of the deflection of the beams. Next, separate coefficients for dead and live load are replaced by combined coefficients for a live load-dead load ratio of 3.0. Then follows a study to determine the effect of discontinuous edges, and this variable is eliminated from the procedure. Finally all slab moments are reduced by 20 percent in recognition of the redistribution of moments at high load. Additional studies are then made of the distribution of moments across the width of the slab and of the moments to be used for the design of the beams to complete the development of the design procedure.

A proposed design procedure is presented in specification form and is compared with other procedures of a similar nature and with the moment distribution procedure described in the appendix.

CORROSION RESISTANCE TESTS OF CONCRETE FLOORS—WITH AND WITHOUT METALLIC AGGREGATE.....45-16

Price 35 cents.

E. W. SCRIPTURE, JR. and C. H. SAKRYD—Dec. 1948 pp. 317-324 (V. 45)

An attempt has been made to determine the conditions under which concrete floors can be expected to give good service. Resistance to abrasion was determined on specimens with and without metallic aggregate after exposure to typical reagents selected from the groups described by the Joint Committee as having corrosive effects of an intermediate nature between no attack and severe attack. On the basis of the experimental results the reagents in the Joint Committee report have been classified in relation to anticipated service of concrete floors.

FLEXICORE PRECAST FLOOR AND ROOF SLABS.....45-17

Price 35 cents.

GAYLE B. PRICE—Dec. 1948, pp. 325-340 (V. 45).

This paper describes briefly the design, manufacture, erection procedures and use of Flexicore slabs in precast concrete floor and roof systems. The Flexicore slab has a standard cross section 12 in. wide and 6 in. thick, lightened by two holes 4⅝ in. in diameter running the full length of the slab which is made in inch variations of length up to a maximum of 22 ft 6 in. Reinforcement consists of two bars in the top of the slab for "handling" stresses and two or three prestressed tension bars at the bottom to take care of bending moment.

PROPOSED RECOMMENDED PRACTICE FOR THE APPLICATION OF PORTLAND CEMENT PAINT TO CONCRETE SURFACES.....45-18

Price 35 cents.

REPORT of COMMITTEE 616—Jan. 1949, pp. 353-368, (V. 45)

This report is a studied revision by a reorganized committee of the previous report of Committee 616 published in June 1942, based upon criticisms of that report plus information from more recent tests and experiences. Included are recommendations for appropriate usage, age of concrete, preparation of surface, and the preparation, application and curing of portland cement paint. Three appendixes discuss composition, manufacture and storage and, general characteristics and factors affecting durability.

DRY MORTAR AS A BEARING AND GROUTING MATERIAL.....45-19

Price 35 cents.

BOYD S. BROOKS—Jan. 1949, pp. 369-380, (V. 45)

In 1939, at the navy's new David Taylor Model Basin at Carderock, Md., there was an urgent need for a non-shrinking bearing material to be used under the cast-steel "chairs" which support the rails on which the towing carriages run. Requests for information concerning materials and installation procedures produced such varied answers that investigation was started to find the best material and the best procedures for placing it.

The bearing material selected for use was 1:3 portland cement mortar, with 4.13 gallons of water per sack of cement. This was rammed firmly into place.

This bearing material has now been in place over a period of about 9 years under extremely severe conditions and without exception has proved to be excellent in function and condition.

DIRECT DIMENSIONING OF RECTANGULAR SECTIONS.....45-20

Price 35 cents.

MICHEL BAKHOUM—Jan. 1949, pp. 381-396 (V. 45)

On the basis of the standard theory for reinforced concrete design, methods have been derived for the direct dimensioning of rectangular sections, with or without compression reinforcement, when subjected to simple bending, eccentric compression or eccentric tension. Two cases are considered: when the depth of the section is fixed and when it is not fixed. The conditions leading to minimum reinforcement are also studied. The solutions developed have been simplified by the use of curves which are applicable for pure bending as well as for eccentric compressive or tensile forces and cover any values of f_c, f_s and n. The curves allow making a variety of designs in a relatively short time and thus help in choosing the most economical or most suitable section.

STRENGTHENING BRIDGE SLABS WITH GROUTED REINFORCEMENT.45-21

Price 35 cents.

S. O. ASPLUND—Jan. 1949, pp. 397-408

On the project described the negative reinforcing bars settled as much as 2½ in. out of correct position. Various means for incorporating negative reinforcement at correct height in the finished structure are discussed. The method selected of grouting additional bars in grooves cut with the aid of diamond saw is described. Tests on beam specimens made with some bars encased in the concrete and some bars grouted into diamond-sawed grooves, give results identical with those to be expected for all bars normally encased; ultimate strengths conform closely to the plastic bending capacity of reinforced concrete according to design methods originated by C. S. Whitney.

COMPARATIVE DESIGNS OF A SEGMENTAL SKEWED FRAME CONCRETE BRIDGE BY THE STRAIGHT LINE AND PLASTIC THEORY METHODS.....45-22

Price 35 cents.

MILTON BRUMER—Jan. 1949, pp. 409-420 (V. 45)

The designs of a segmental skewed frame concrete bridge are discussed and compared as developed by both the straight line and plastic theory methods. The plastic theory will lead to rigid concrete frames of more economical proportions capable of sustaining equal or greater loads than frames designed by the straight line method. Effects of volume changes and yielding supports are also of smaller consequence. Deflections may be expected to be greater. The author concludes that further substantiation of the plastic theory is desirable.

SOME FACTORS AFFECTING AIR ENTRAINMENT.....45-23

Price 35 cents.

E. W. SCRIPTURE, Jr. and F. J. LITWINOWICZ—Feb. 1949, pp. 433-444 (V. 45).

A program was undertaken to investigate the influence of various factors on the amount of air entrained in concrete mixes with and without air-entraining agent added. The variables so far studied are slump, cement factor, and sand-total aggregate ratio. The results indicate that the two latter factors are of major importance but that slump has less effect. In general, entrained air content increases with increasing slump, decreases with increasing cement factor, sand-total aggregate ratio.

EXPERIMENTAL AIDS IN STRUCTURAL CONCRETE DESIGN...45-24

Price 35 cents.

R. E. GLOVER, O. J. OLSEN and CARL ZANGAR—Feb. 1949, pp. 445-468 (V. 45).

Statically determinate and indeterminate structures are generally designed by analytical methods. This paper describes several experimental aids which have been used in the design and stress analysis of statically indeterminate structures at the laboratories of the Bureau of Reclamation. These methods are used where analytical procedures cannot conveniently be applied. Described herein are photoelastic methods using both the polariscope and interferometer, the Beggs Deformeter, Stresscoat and electric SR-4 strain gages. Applications are given for each method.

PRACTICES, EXPERIENCES, AND TESTS WITH AIR-ENTRAINING AGENTS IN MAKING DURABLE CONCRETE.....45-25

Price 35 cents.

R. F. BLANKS and W. A. CORDON—Feb. 1949, pp. 469-488 (V. 45)

The authors discuss the advantages of air entrainment with regard to durability, permeability, workability, reduction in alkali expansion, time saving, reduction in water and cement, temperature rise, strength, abrasion resistance, monolithic lightweight concrete and mass concrete. U. S. Bureau of Reclamation experiences are discussed with regard to pumping, transportation and placing, grading of aggregates, factors affecting the amount of air entrained Air-entraining agents and air-entraining cements are discussed briefly. There is a brief section on the measurements of entrained air in concrete with a recommended procedure.

CONCRETE FLOOR FINISHING.....45-26

Price 35 cents.

GERALD MILSOM—Feb. 1949, pp. 489-492 (V. 45).

A concise discussion of the requirements for durable concrete floors including careful selection and control of materials and attention to weather conditions. The importance of proper equipment, working conditions and curing of the finished concrete are emphasized.

PLASTIC FLOW OF CONCRETE AT HIGH OVERLOAD.....45-27

Price 35 cents.

J. R. SHANK—Feb. 1949, pp. 493-500 (V. 45).

New data on the plastic flow of concrete prompted a reconsideration of the assumption that plastic flow is directly proportional to the unit stress applied. Tests are described in which deformations of 6 x 12-in. concrete cylinders were measured while under sustained loads approaching the ultimate strength. Plotted data are shown to demonstrate that the change in the rate of plastic flow appears to be uniform up to three-quarters or more of the ultimate strength with an abrupt change at the "true" ultimate strength. Included are other data indicating that the plastic flow curve has the same general form independent of time and applied load leading to a new consideration in expressing factor of safety and developing beam theories on a factor of safety basis.

CONTROL OF CONCRETE PAVEMENT SCALING CAUSED BY CHLORIDE SALTS.....45-28

Price 35 cents.

B. D. TALLAMY—Mar. 1949, pp. 513-520 (V. 45)

Under modern traffic requirements on heavily traveled roads salt-treated abrasives will not remove ice quickly enough. As maintenance forces have struggled to meet the demand for uninterrupted service the use of straight salts has become increasingly common. Direct applications of up to 600 lb of salt per two-lane mile are required to combat extreme icing conditions. Pavements constructed to withstand the weak brines deposited by salt-treated abrasives cannot stand up under straight salt. New pavements appear more vulnerable than those two or more years old.

The observed resistance to salt action of the oil soaked center streak of uphill traffic lanes led New York state highway engineers to investigate the feasibility of the use of dilute oil applications as a protective measure. Laboratory tests indicate successful results may be expected. In the late summer of 1948 oil spray equipment was developed in time to treat nearly 60 miles of new concrete highways, which should provide a wide base for field observation of the effectveness of the method.

EXPERIENCE WITH AIR-ENTRAINING CONCRETE IN NEW JERSEY...45-29

Price 35 cents.

CHARLES M. NOBLE—Mar. 1949, pp. 521-528 (V. 45)

This paper outlines experience in New Jersey with air-entraining concrete utilized primarily to combat attack by de-icing chemicals. Damage to concrete pavements in New Jersey usually is associated with heavy loads. Freedom from disintegration troubles, except in isolated cases, is attributed to excellent aggregates, rigid laboratory control, mix design, inspection of material and plant equipment and construction supervision at the job site.

Air-entraining portland cement concrete was first specified for a concrete pavement contract in 1945 and since then has been used on many contracts with notable success to prevent deterioration due to scaling. The same high standards for materials, mixture design, laboratory control, field inspection and attention to details must be maintained as with ordinary concrete. Pavements thus far constructed in New Jersey with air entrainment have not

scaled or shown any signs of disintegration when ice control chemicals have been properly mixed with abrasives at a rate not exceeding 75 lb per cu yd of abrasive. It is too early to form judgment but indications are the results achieved fully justify the use of air-entraining concrete.

DESIGN DETAILS FOR ARCHITECTURAL CONCRETE.....45-30

Price 35 cents.

J. J. HOGAN—Mar. 1949, pp. 529-540 (V. 45)

"To achieve beauty with concrete, as with any other material, requires studied design and skillful craftsmanship" are the words of architect Robert D. Murray. This paper directs attention to the matter of "studied design". The skilled craftsmanship required to construct architectural concrete must be preceded by proper design by the architect and engineer. The designer must understand and appreciate the fundamental requirements of this type of construction. This paper discusses considerations appropriate to achieving the desired texture of surface as affected by form materials selected. Ornamentation should be considered in every detail from the standpoint of practicability. Construction joints are inevitable and should be specified in logical locations. Similarly, control joints should be specified with an understanding of where the most likely cracks due to volume change will occur and the design of reinforcement should be coordinated with the location of the control joints. Numerous illustrations support the text.

CONSTRUCTION PRACTICES FOR ARCHITECTURAL CONCRETE.....45-31

Price 35 cents.

E. B. OBERLY—Mar. 1949, pp. 541-552 (V. 45)

Detailed plans and carefully drawn specifications executed by a contractor and superintendent with the know-how of form construction are necessary for good architectural concrete. This paper describes some of the more important construction practices to be considered to achieve the results desired. Form details and concrete quality are discussed in considerable detail, the former being supported by illustrations.

CONCRETE OF THE FUTURE 45-32

Price 35 cents.

ROBERT F. BLANKS—April 1949, pp. 565-568, (V. 45)

Mr. Blanks in his remarks as retiring president, depicts what concrete might be 50 years in the future and discusses the role of the engineer in that era.

CONTROL OF QUALITY OF READY MIXED CONCRETE 45-33

Price 35 cents.

STANTON WALKER—Apr. 1949, pp. 569-580 (V. 45)

Control of quality of ready-mixed concrete is discussed. The objective of the paper is to pose the more important problems of concrete control, especially those peculiar to ready-mixed concrete, with the view of eliciting discussion, rather than to attempt to outline recommended practices. Both central-mixing and transit-mixing are discussed. Attention is directed to the mutuality of responsibility of the producers and users in bringing about effective control.

TESTS OF LIGHTWEIGHT AGGREGATE CONCRETE DESIGNED FOR MONOLITHIC CONSTRUCTION....45-34

Price 35 cents.

WALTER H. PRICE and WILLIAM A. CORDON—Apr. 1949, pp. 581-600 (V. 45)

The characteristics and origin of lightweight aggregates are described and the comparative results of laboratory tests of strength, insulating value, shrinkage, and weathering resistance of concrete made with light weight aggregates from 17 producers are summarized. Data are included on expanded shale, expanded slag, scoria, pumice, perlite, exfoliated vermiculite and diatoamceous earth.

WHAT HAVE WE LEARNED ABOUT AIR-ENTRAINING CONCRETE?.....45-35

Price 35 cents.

J. F. BARBEE—Apr. 1949, pp. 601-612 (V. 45)

A review of ACI published data on air-entraining concrete, this paper summarizes methods of control and the effect of air entrainment on durability, plasticity and workability, strength, resistance to abrasion and other properties of concrete. The effects of variations in cement and water content, sand-coarse aggregate ratio, mixing time and the use of calcium chloride as an admixture are described. Changes in technique necessary to successfully place, vibrate and finish air-entraining concrete are pointed out. A detailed bibliography follows.

DEVELOPMENTS IN THE MANUFACTURE AND TECHNOLOGY OF CONCRETE MASONRY UNITS.....45-36

Price 35 cents.

JAY C. EHLE—Apr. 1949, pp. 613-620 (V. 45)

The author touches the high spots of developments in the manufacture of concrete masonry units listing the many problems which now face the manufacturer. Rapid growth of the industry in the past ten years and the change from tamping to vibration methods is described. New plants are notable for labor saving equipment compared with those of the middle 1930's. In addition to block manufacture and several methods of curing, in use and proposed, the author mentions briefly: block-handling equipment, maintenance and repair of the plant, and the greater supervisory skill now required for successful operation.

NOTICE!

Copies of original books and magazine articles reviewed in the Current Reviews section of the JOURNAL are **not** available from ACI. The footnote formerly published with that section indicated that separate prints of *the reviews section itself* are available, not separate prints of the individual articles reviewed, as some readers have assumed.

DISCUSSION

Discussion closed January 1, 1949

Sept. Jl. '48

Recommended Practice for Winter Concreting Methods (ACI 604-48)—Report of Commitee 604

Relative Strengths of Portland Cement Mortar in Bending Under Various Loading Conditions—Jacob Porter Frankel

Terrazzo as Affected by Cleaning Materials—D. W. Kessler

Transporting Ready-Mixed Concrete in Open Dump Trucks—R. A. Burmeister

A Study of Alkali-Aggregate Reactivity by Means of Mortar Bar Expansions—T. M. Kelly, L. Schuman, F. B. Hornibrook

Discussion closed February 1, 1949

Oct. Jl. '48

Lattice Analogy in Concrete Design—Douglas McHenry

Strength of Precast Concrete Floor Joists—Jacob Feld

Effect of Entrained Air on Concretes Made With So-Called "Sand-Gravel" Aggregates—Paul Klieger

Burned Shale and Expanded Slag Concretes With and Without Air-Entraining Admixture—P. H. Petersen

Discussion closed March 1, 1949

Nov. Jl. '48

Reinforced Concrete Wall and Column Footings, Parts 1 and 2—Frank E. Richart

Cemenstone Precast Construction—A. C. Grafflin

Sand Grading Influence on Air Entrainment in Concrete—M. A. Craven

Influence of Size Grading of Sand on Air Entrainment—E. W. Scripture, Jr., F. B. Hornibrook and D. E. Bryant

Use of Prepacked Aggregate Concrete in Major Dam Construction—Paul Baumann

Discussion closed April 1, 1949

Dec. Jl. '48

Rational Analysis and Design of Two-Way Concrete Slabs—C. P. Siess and N. M. Newmark

Corrosion Resistance Tests of Concrete Floors—With and Without Metallic Aggregate—E. W. Scripture, Jr. and C. H. Sakryd

Flexicore Floor and Roof Slabs—Gayle B. Price

Discussion closes May 1, 1949

Jan. Jl. '49

Proposed Recommended Practice for the Application of Portland Cement Paint to Concrete Surfaces—Report of Committee 616

Dry Mortar as a Bearing and Grouting Material—Boyd S. Brooks

Direct Dimensioning of Rectangular Sections—Michel Bakhoum

Strengthening Bridge Slabs with Grouted Reinforcements—S. O. Asplund

Comparative Designs of a Segmental Skewed Frame Concrete Bridge by the Straight Line and Plastic Theory Methods—Milton Brumer

Discussion closes June 1, 1949

Feb. Jl. '49

Some Factors Affecting Air Entrainment—E. W. Scripture, Jr. and F. J. Litwinowicz

Experimental Aids in Structural Concrete Design—R. E. Glover, O. J. Olsen and Carl Zangar

Practices, Experiences and Tests with Air-Entraining Agents in Making Durable Concrete—Robert F. Blanks and W. A. Cordon

Concrete Floor Finishing—Gerald Milsom

Plastic Flow of Concrete at High Overload—J. R. Shank

Discussion closes July 1, 1949

Mar. Jl. '49

Control of Concrete Pavement Scaling Caused by Chloride Salts—B. D. Tallamy

Experiences With Air-Entraining Concrete in New Jersey—Charles M. Noble

Design Details for Architectural Concrete—John J. Hogan

Construction Practices For Architectural Concrete—E. B. Oberly

Discussion closes August 1, 1949

Apr. Jl. '49

Concrete of the Future—Robert F. Blanks

Control of Quality of Ready-Mixed Concrete—Stanton Walker

Tests of Lightweight-Aggregate Concrete Design for Monolithic Construction—Walter H. Price and William A. Cordon

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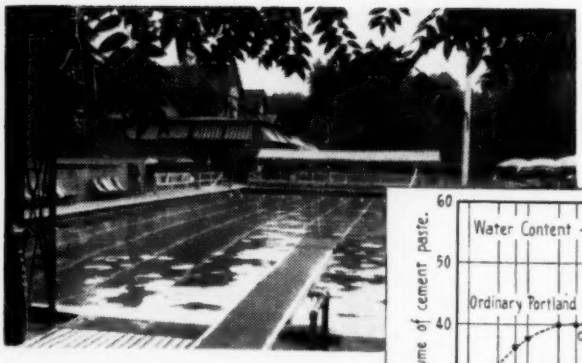

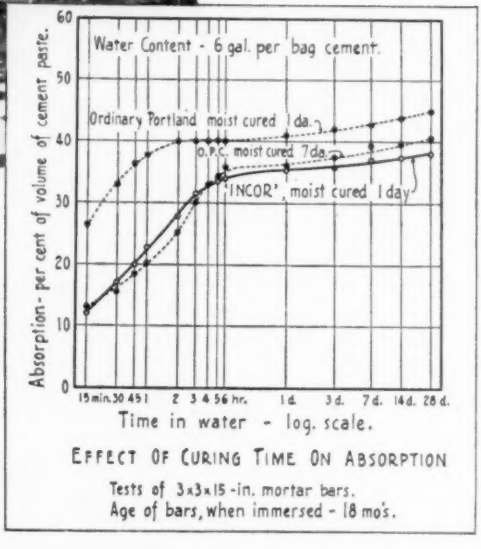
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Vol. 20—No. 9 May 1949 (Proceedings Vol. 45)

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POLICY

It is the policy of the American Concrete Institute to encourage participation by its members and others in the work of extending the knowledge of concrete and reinforced concrete as a basis for improved products and structures.

To this end the Board of Direction has assigned to the Technical Activities Committee the responsibility of selecting for publication such papers, committee reports, discussions and other contributions or parts of such contributions as, in the judgment of the Committee, seem to offer most of value in attaining Institute objectives within space requirements, consistent with budget limitations.

"Lightweight-Aggregate Concrete," p. 625, reports a National Bureau of Standards study of lightweight aggregates, most of them available in eastern United States, to determine their physical properties and behaviour in plastic concrete. This investigation, with another by the U. S. Bureau of Reclamation on western lightweight aggregates (See April JOURNAL), was made for the National Housing and Home Finance Agency to provide basic information on these materials for those contemplating their use in building construction.

Discussion is a forum for opinions concerning the subject matter of JOURNAL papers. To permit publication while information is still current requires a time limitation on submitting discussion. Please note the closing date—Sept. 1, 1949—for papers appearing in this issue.

Separate prints of all JOURNAL papers are available at time of publication at nominal prices and are kept in stock as long as the supply lasts. Over 90 percent are available back to 1937. Consult your 10-year index for subject matter and titles. Missing title numbers are available in "in stock" back issues of the JOURNAL. Any paper not in stock can be reproduced for large quantity orders. Inquiries as to cost should be addressed to the Secretary-Treasurer.

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In ACI Journal next month

The application of a relatively new research tool tt cement hydration by GEORGE L. KALOUSEK, CURTIS W. DAVIS and WILLIAM E. SCHMERTZ to identify the hydrous solids formed in cements and to partially follow the course of some of the hydration reactions form the background for "An Investigation of Hydrating Cements and Related Hydrous Solids by Differential Thermal Analysis."

FRANK R. BEYER reports a study of "Stresses in Reinforced Concrete due to Volume Change" based on the use of SR-4 strain gages attached to reinforcing steel before specimens were poured. The definite cycle of stresses recorded from the time of pouring until the concrete was three months old indicate that these stresses should be considered in design.

The elements of success in producing good concrete floors are given by MORGAN B. KLOCK in "Monolithic and Bonded Floor Finishes." In brief form he describes the desirable characteristics of materials and proper construction technique.

How site difficulties led to the use of pumped concrete is described by C. O. CRANE in "Concrete for Enders Dam Spillway Placed by Pumping Method." He describes the characteristics of the concrete used, the pipeline layout and difficulties encountered and overcome in the course of the job.

Discussions of several ACI JOURNAL papers are scheduled for June. Included will be discussion by J. TUCKER, JR. of JACOB FRANKEL'S paper "Relative Strengths of Portland Cement Mortar in Bending Under Various Loading Conditions," and discussion by BRYANT MATHER and T. E. STANTON of "A Study of Alkali-Aggregate Reactivity by Means of Mortar Bar Expansions," by T. M. KELLY, L. SCHUMAN and F. B. HORNIBROOK.

Title No. 45-37

The critical housing shortage together with the scarcity of materials and the desire for better thermal insulation have resulted in the introduction and use of many new lightweight aggregates. Data are reported here on the characteristics of some of these materials as constituents of plastic concrete.

Lightweight-Aggregate Concrete*

By RALPH W. KLUGE†, MORRIS M. SPARKS‡, and EDWARD C. TUMA§

SYNOPSIS

Results of studies on lightweight-aggregate concretes made by the National Bureau of Standards at the request of the Housing and Home Finance Agency (formerly the National Housing Agency) are reported in this paper. Aggregates studied were expanded clay, shale, and slate; three types of expanded blast-furnace slag; expanded vermiculite and perlite; sintered diatomite and fly ash; and a pumice. Concretes of various proportions were made from these aggregates and determinations were made of the weight per cubic foot, compressive and transverse strength, shrinkage, elastic modulus, absorption, resistance to freezing and thawing and thermal conductivity. Test results are briefly discussed and summarized.

INTRODUCTION

In recent years a wide variety of lightweight materials, both natural and processed, have been introduced and marketed for use as concrete aggregate. The advantage of concrete of light weight is obvious, but perhaps equally as important are the favorable thermal insulating properties exhibited by such concretes. However, the little technical data available concerning the structural and thermal insulating properties of lightweight-aggregate concrete are widely scattered and, for the most part are confined to expanded shales, expanded slags and cinder concretes.

Recognizing the need for information that would disclose the possible usefulness of lightweight concrete for housing, the Housing and Home Finance Agency (formerly the National Housing Agency) requested the National Bureau of Standards and U. S. Bureau of Reclamation to test

*Presented at ACI 45th annual convention, New York, N. Y. Feb. 25, 1949. Title No. 45-37 is a part of copyrighted JOURNAL of the AMERICAN CONCRETE INSTITUTE, V. 20, No. 9, May 1949, *Proceedings* V. 45. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Sept. 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

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‡Hydraulic Engineer, Beach Erosion Board, Corps of Engineers. Formerly Engineer, National Bureau of Standards, Washington, D. C.

§Engineer, National Bureau of Standards, Washington, D. C.

concretes made with commercially available lightweight aggregates for the purpose of providing information for designers, engineers and architects.

The tests conducted at the National Bureau of Standards, and reported in this paper, were confined to aggregates available principally in the eastern part of the United States, and, except for expanded slag, were limited to single samples of aggregate representative as to type of material. The investigation was confined to aggregates of mineral origin and further limited to plastic concrete as distinguished from pressed or tamped concrete used in concrete masonry units.

The eleven lightweight aggregate samples included in the study were exfoliated vermiculite, sintered diatomite and fly ash, expanded perlite, blast-furnace slags, shales, slates, and clays; and one natural material, pumice. A brief description of each aggregate is given in Table 1. Two of these samples were furnished for experimental purposes by the Bureau of Mines, College Park Station, Maryland; the other nine were commercial products. For comparison, Potomac River sand and gravel were also included and treated in the same manner as the lightweight aggregates.

The study consisted of two parts: (1) tests concerned with the properties of the aggregates themselves and (2) tests to determine the properties of concrete containing various proportions of cement and aggregates.

The aggregate tests: sieve analyses, specific gravity, absorption, unit weight, grading (mechanical analysis) and crushing strength.

The concrete tests: compressive strength, flexural strength, shrinkage, absorption, elastic modulus (both static and dynamic), resistance to freezing and thawing, and thermal conductivity.

The proportions of cement to aggregate were such as to produce concrete mixtures of each aggregate containing nominally 3, 5, 7 and 9 bags of cement per cu yd of concrete.

CHARACTER OF THE AGGREGATES AND THEIR EFFECT ON PROPORTIONING AND MIXING

Aggregate tests

The rough vesicular surface character of most of the lightweight aggregates studied created special problems in testing the aggregates themselves as well as in mixing and placing the concrete containing them.

Special testing techniques were required to determine the specific gravity and absorption of the aggregates. Both of these properties depended upon obtaining a sample of material in a saturated surface-dry condition which required considerable skill and judgment on the part of the operator. Whereas similar tests of natural dense aggregates

TABLE 1—DESCRIPTION OF LIGHTWEIGHT AGGREGATES TESTED

Symbol	Aggregate	Description
V	Exfoliated vermiculite	A type of mica expanded by heat. Soft fragile particles of flaky laminated structure. Color—brown to buff with metallic luster on laminated surfaces.
D	Sintered diatomite	A processed diatomite which had soft chalky particles. Color—light tan to light pink.
P	Perlite	An expanded perlite composed of frothy fragile particles of irregular shape. Color—white.
S 1	Expanded slag No. 1	A slag expanded in the pit process. Particles were irregular, the relatively large cell structure creating a rough surface texture. Color—light to dark gray.
S 2	Expanded slag No. 2	A slag expanded from runner. Similar in surface characteristics to expanded slag No. 1. Color—almost black to light gray with occasional cream-colored particles.
S 3	Expanded slag No. 3	A machine-processed expanded slag. Similar in appearance to expanded slag No. 2. Color—uniformly cream colored with only occasional gray particles.
F A	Sintered fly ash	A processed material resulting from the combustion of powdered fuel in steam power plants. Similar in appearance to ordinary cinders, extremely rough and irregular in shape. Numerous black glass particles. Color—dark brown to black.
Pu	Pumice	A natural lightweight material of volcanic origin. Particles had uniformly small cell structure with fairly soft surfaces of granular texture. Color—off white.
E Sh	Expanded shale	A shale expanded by heat. Particles were angular but not sharp or rough. Internal structure was smaller, with more uniformly sized cells than the expanded slags. Color—uniformly medium gray.
E Sl	Expanded slate	A slate expanded by heat with fairly well-rounded and reasonably smooth particles of laminar structure. Color—buff to brown. Material produced experimentally by the Bureau of Mines, College Park Station.
E C	Expanded clay	A clay expanded by heat. Well-rounded and fairly smooth surface and uniformly small cell structure. Color—tan to gray. Material produced experimentally by the Bureau of Mines, College Park Station.

are reproducible to less than ½ percent, reproducibility with identical samples of lightweight aggregates was about 4 or 5 percent.

Table 2 lists the results of the aggregate tests as obtained either by special procedures or standard methods where applicable.

Also included in the table are the oven-dry unit weights of the aggregates obtained by two methods and the crushing strength of each material.

TABLE 2—PHYSICAL PROPERTIES OF LIGHTWEIGHT AGGREGATES

Aggregate	Unit weight, lb per cu ft		Bulk specific gravity[1]	Absorption		Crushing strength, lb per sq in.	
	Rodding method	Jigging method		By weight, (percent)	By volume, (lb H_2O per cu ft)	1-in. compaction	2-in. compaction
Exfoliated vermiculite	10.0	9.3	1.35	128.5	18.5	11	38
Sintered diatomite	31.0	33.6	1.44	75.2	23.8	615	3,840
Perlite	9.4	9.2	0.86	153.2	14.3	30	91
Expanded slag No.1	67.0	71.9	2.36	9.3	6.7	395	1,360
Expanded slag No.2							
Fine	61.0	70.5	2.26	8.0	5.6	240	1,040
Coarse	48.0	50.1	1.96	22.8	10.4		
Expanded slag No.3							
Fine	59.0	59.4	2.08	5.2	2.9	360	1,290
Coarse	31.0	41.1	1.13	21.3	7.1		
Sintered fly ash:							
Fine	58.0	65.7	2.10	8.2	1.4	325	1,160
Coarse	42.0	44.8	1.74	15.2	6.3		
Pumice:							
Fine	49.0	53.2	1.66	29.1	14.9	485	1,710
Coarse	44.0	33.1	1.46	42.8	16.9		
Expanded shale:							
Fine	76.0	78.6	2.09	7.5	5.7	2120	16,500
Coarse	60.0	64.7	1.74	6.9	4.6		
Expanded slate:							
Fine	73.0	72.4	2.20	8.2	7.1	635	2,120
Coarse	39.0	40.8	1.29	15.0	6.1		
Expanded clay:							
Fine	66.0	70.5	1.98	15.8	11.1	935	6,930
Coarse	53.0	54.6	1.65	21.2	11.6		

[1]Determined on saturated surface-dry aggregate (ASTM C 128-42).

Gradation

In general, lightweight aggregate mixtures required a much higher percentage of material passing a No. 4 sieve than sand-gravel aggregates to produce a reasonably workable concrete. Fig. 1 shows the gradation curves for a typical lightweight aggregate containing fine and coarse material as compared with a normal gradation of a Potomac River sand and gravel mixture. The percentage of fine material depended upon the surface character of the aggregate and on the maximum size of the coarse aggregate used. There was, however, a limit to the proportion of fine material that could be used without adversely affecting the strength of the concrete for any given cement content.

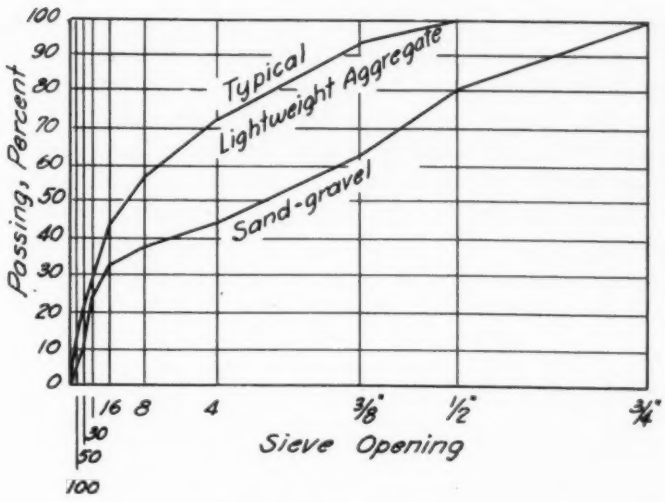


Fig. 1—Gradation curves

TABLE 3—SIEVE ANALYSES OF LIGHTWEIGHT AGGREGATES

Aggregate	Percent aggregate passing U. S. standard sieve									
	1 in.	¾ in.	½ in.	⅜ in.	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100
Exfoliated vermiculite	...	...	...	100	99	95	77	38	7	1
Sintered diatomite	...	...	...	100	84	52	29	14	5	2
Perlite	...	...	...	...	100	96	73	39	20	12
Expanded slag No. 1	...	100	99	96	85	71	53	35	19	9
Expanded slag No. 2:										
Fine	...	...	...	100	96	77	52	30	17	10
Coarse	...	...	100	81	13	7	6	6	5	4
Expanded slag No. 3:										
Fine	...	...	...	100	100	85	63	37	18	11
Coarse	...	...	100	79	19	10	9	9	8	7
Sintered fly ash:										
Fine	...	...	...	100	85	58	37	24	15	10
Coarse	...	100	91	58	11	8	8	7	7	5
Pumice:										
Fine	...	...	...	100	89	71	54	40	30	21
Coarse	100	93	64	48	34	25	20	16	12	9
Expanded shale:										
Fine	...	...	...	...	100	99	71	46	31	20
Coarse	...	...	100	84	11	2	2	1	1	1
Expanded slate:										
Fine	...	...	...	...	100	90	61	38	21	10
Coarse	...	...	100	99	55	11	5	3	2	2
Expanded clay:										
Fine	...	...	...	...	100	87	61	46	38	26
Coarse	...	100	99	80	48	28	15	12	10	6

The gradations of each aggregate as used in the concrete mixtures tested are given in Table 3. These gradations were not necessarily the best from either the standpoint of workability or strength of the concrete produced. Because of the limited quantity of each material available for study, screening and recombining of the various sizes for the purpose of securing the best gradation was not feasible. In lieu of such a procedure, a gradation which seemed reasonable was established by combining various proportions of fine and coarse material where such sizes were supplied separately. The final gradation was therefore governed largely by the individual gradations of the fine and coarse materials.

Workability

Air entrainment was found to assist materially in promoting workability of the concrete. It tended to prevent segregation of the cement from the lightweight aggregate particles, reduce the water requirement and, of course, further reduced the weight of the concrete. An example of the effect of air entrainment on a harsh mixture is illustrated in Fig. 2. The two samples of fresh concrete shown contained identical proportions of cement and aggregate and the same quantity of mixing water. The sample on the left contained, in addition, approximately 0.05 percent Vinsol N.V.X. by weight of cement. The cohesiveness and the ability of the mixture which contained the air-entraining agent to retain water was evident, whereas serious segregation and bleeding were apparent in the other mixture.

Little is known, however, regarding the limitations of air entrainment or with what gradations it can be employed most effectively from the

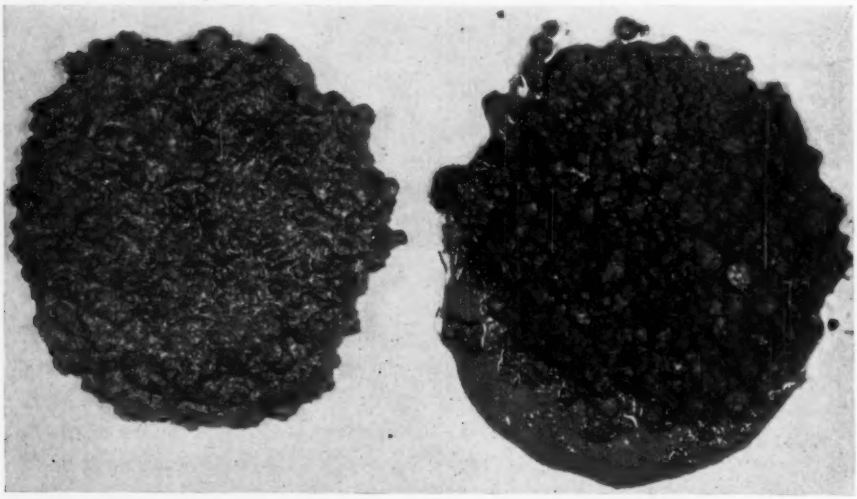


Fig. 2—Concrete on left shows effect of air entrainment on harsh mixture

standpoint of both workability and strength. Such a study was outside the scope of this investigation. There was some evidence to indicate that up to a certain percentage of air, a value apparently somewhat higher than normally permitted for dense aggregate concrete, strength of the lightweight concrete was improved, undoubtedly because of the reduction in water required and the absence of severe segregation that is so often evident in harsh mixtures of lightweight aggregates such as the one shown in Fig. 2.

Consistency

With air entrainment, a consistency corresponding to a slump of 4 or 5 in. was found to be satisfactory for casting specimens. Some difficulty was experienced in controlling this consistency with certain aggregates because they were used in a dry condition. Continued absorption of the mixing water was responsible for this difficulty. Many of the aggregates absorbed most of their water during the mixing period. However, some continued to absorb appreciable quantities of water even after 4 or 5 minutes of mixing, thus drying the mixture during the placing operations. In effect this amounted to lowering the water-cement ratio and better strengths might be expected than with similar concretes which contained presoaked aggregates. On the other hand, obtaining concretes with good consistency and workability might be a problem with these absorptive aggregates.

Proportioning and mixing

The concrete mixtures were proportioned by volume expressed as one part of cement to a given number of parts of dry aggregates including fine and coarse material. Due to the wide differences in the rates of absorption and total absorption of the various-sized grains of the aggregates the water-cement ratio method was impracticable for designing the mixes. Water requirement of the mix was therefore based on a given consistency as measured by the slump cone, although this device was, admittedly, not altogether satisfactory for lightweight-aggregate concrete.

Although the mixtures were proportioned by volume the batches of materials were weighed. The quantities were such as to yield about 2 cu ft of concrete per batch, mixed in a 3-cu ft tilt-drum mixer. All of the aggregates were used in a surface-dry condition. They were first mixed for a period of one minute with water in which the Vinsol resin had been added. Cement was then added and mixed for an additional 3 minutes. The only departure from this order of mixing was with the vermiculite aggregate. A slurry of cement, Vinsol resin, and water was first mixed for one minute, then the exfoliated vermiculite was added and mixed for 3 minutes.

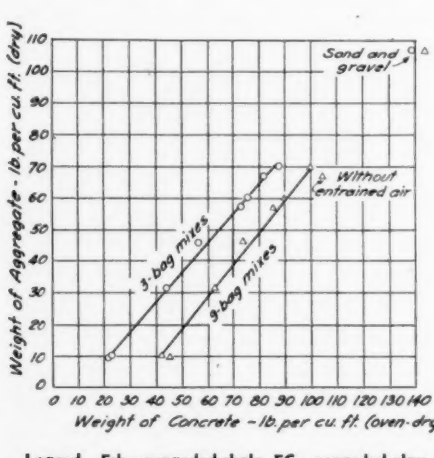
PROPERTIES OF CONCRETE

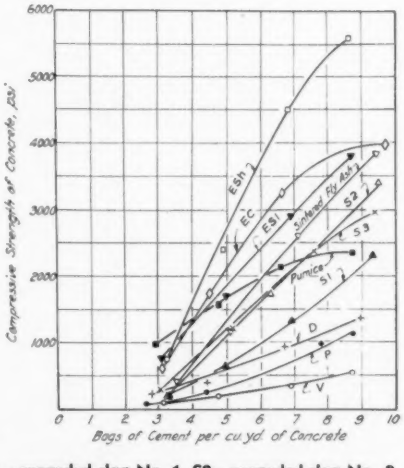
Weight and strength

Unit weights of the lightweight concretes ranged from 50 to 112 lb per cu ft fresh and from 20 to 105 lb per cu ft oven dried. The weights for each mix and for each type of concrete are listed in Table 4. Relations between the unit weight of the aggregates (combined fine and coarse) and the unit weight of the concretes made therefrom are shown in Fig. 3 for the leanest and richest mixtures. The weight of the concrete is, of course, influenced by the quantity of air entrained. The one mix in which air was not entrained is seen to depart from the general trend of values. Also shown in the figure are values for sand-gravel concrete.

High early strength cement was used in all of the concrete mixtures, consequently a large proportion of the maximum compressive strength was developed in the first 7 days. The relationship between the compressive strength, obtained from 6 x 12-in. cylinders, and the cement content of each concrete is shown in Fig. 4 and 5 for 7-day and 28-day periods respectively. Individual values are given in Table 4. All concrete specimens were cured for 7 days in a fog room.

The relation between weight per cu ft and compressive strength for each of the concretes and the relation between these properties for all lightweight aggregate concretes is shown in Fig. 6. It is apparent from the figure that for a given compressive strength there was a wide range in weight among concretes produced with various aggregates. For example, the range is from 50 to 95 lb per cu ft for a strength of 1000 psi. It may be noted that those aggregates which were lighter in weight usu-


Weight of Aggregate - lb. per cu. ft. (dry)
Sand and gravel
Without entrained air
3-bag mixes
3-bag mixes
Weight of Concrete - lb. per cu. ft. (oven-dry)



Legend: Esh—expanded shale, EC—expanded clay, S1—expanded slag No. 1, S2—expanded slag No. 2, S3—expanded slag No. 3, ESl—expanded slate, D—sintered diatomite, P—perlite, V—exfoliated vermiculite

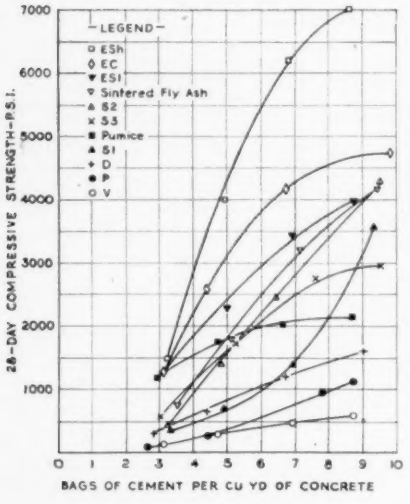
Fig. 3 (left)—Weight of concrete compared to weight of aggregate

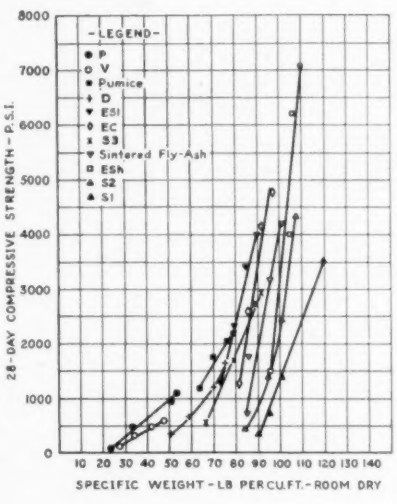
Fig. 4 (right)—Relationship between 7-day compressive strength and cement content

TABLE 4—PHYSICAL PROPERTIES OF LIGHTWEIGHT-AGGREGATE AND SAND-GRAVEL CONCRETES

Aggregate	Cement–Aggregate ratio by volume	Proportions by volume			Bags of cement per cu yd concrete (avg. of 3 mixes)	*Percent V/sol N.V.K.	Cu yd of Aggregate per cu yd of concrete		Slump, in. (avg. of 3 mixes)	Water, * * gals per cu yd conc (avg. of 3 mixes)	Wt. of concrete lbs. per cu ft. (avg. of 3 spec)			Percent Absorption (avg. of 3 spec)		Compressive Strength, psi (avg. of 5 spec.)			Transverse Strength, psi (avg. of 3 spec)		Elastic Modulus (avg. of 3 spec)		Thermal *Conductivity	Durability Factor (avg. of 3 spec)	Shrinkage at 100 days (in. per ft.) (avg. of 3 spec)	Concrete Workability
		Cement	Fine	Coarse			Fine	Coarse			Fresh	Room Dry	Oven Dry	by Weight	by Volume	7 days	28 days	Oven dry	28 days	Oven dry	Static x 1000	Sonic x 1000				
V	1:9.4	1		9.4	3.1	0.06		1.10	6.0	110	50	28	23	117.9	5.1.0	80	165	165	38	36	86	92	0.97	4	0.0415	Good
	1:5.3	1		5.3	4.7	0.06		0.94	7.0	99	54	34	29	85.2	44.2	185	290	325	76	72	158	161	1.02	10	0.0565	Very Good
	1:3.9	1		3.9	6.9	0.06		1.01	6.0	105	59	42	36	51.5	33.6	325	470	525	43	46	181	187	1.41	43	0.0510	Very Good
	1:2.9	1		2.9	8.7	0.06		0.90	6.0	104	63	49	42	35.9	26.9	540	585	715	67	108	173	263	1.60	66	0.0475	Very Good
D	1:9.0	1		9.0	2.8	0.04		0.94	4.0	116	73	51	43	69.3	53.6	235	320	281	58	34	221	255	1.16	† 1	0.0315	Fair to Good
	1:6.0	1		6.0	4.4	0.04		0.97	4.0	121	77	58	48	60.1	52.3	400	654	500	70	44	282	360	1.27	† 2	0.0315	Fair to Good
	1:3.7	1		3.7	6.7	0.04		0.91	4.0	125	85	70	58	48.2	49.3	940	1200	1040	86	55	354	388	1.56	† 2	0.0310	Good
	1:2.4	1		2.4	9.0	0.04		0.80	5.0	122	91	76	62	39.2	44.0	1365	1625	1490	(55)	67	326	426	1.80	† 2	0.0415	Good
P	1:10.3	1		10.3	2.6	0.05		1.03	3.0	92	43	24	21	94.5	32.5	50	95	82	42	(36)	84	98	0.75	18	——	Fair to Good
	1:5.5	1		5.5	4.4	0.05		0.90	7.0	74	44	33	28	54.4	27.3	250	270	267	(53)	(50)	120	157	0.92	78	0.0235	Good
	1:3.4	1		3.4	7.8	0.03		0.99	4.0	72	55	51	44	29.1	22.0	975	950	876	(69)	(53)	272	239	1.39	99	0.0275	Good
	1:2.9	1		2.9	8.7	0.03		0.93	5.0	74	58	53	46	26.4	21.1	1100	1120	962	(62)	(67)	290	279	1.45	77	0.0230	Very Good
S1	1:7.6	1		7.6	3.3	0.03		0.94	4.0	81	97	90	81	20.0	28.4	207	370	410	127	140	733	900	2.01	27	0.0200	Poor to Fair
	1:5.0	1		5.0	4.9	0.02		0.91	4.0	78	99	96	86	17.6	26.4	610	705	915	177	227	1285	1372	2.42	76	0.0110	Fair to Good
	1:3.5	1		3.5	6.9	0.02		0.89	5.0	75	103	101	90	14.5	22.6	1300	1400	1880	328	420	1789	1891	2.92	92	0.0150	Very Good
	1:2.8	1		2.8	9.3	0		0.97	5.0	89	120	119	105	13.8	25.2	2295	3565	3600	460	483	2630	2655	3.16	20	0.0150	Good
S2	1:8.9	1	5.80	3.10	3.2	0.05	0.68	0.42	5.0	59	66	84	76	17.7	23.3	154	425	380	121	140	733	856	1.90	39	0.0125	Fair to Good
	1:6.6	1	4.35	2.25	4.8	0.03	0.77	0.45	3.0	67	98	97	89	14.5	23.3	530	1400	1280	278	286	1560	1706	2.49	69	0.0100	Fair to Good
	1:4.7	1	3.05	1.65	6.4	0.02	0.73	0.41	5.0	68	105	101	94	13.3	21.3	1700	2450	2300	365	440	2330	2052	2.84	77	0.0110	Fair to Good
	1:3.0	1	1.95	1.05	9.5	0.02	0.68	0.36	5.0	72	112	108	101	10.8	18.6	3360	4300	4070	412	519	2390	2551	3.16	68	0.0125	Good
S3	1:9.0	1	4.50	4.50	3.0	0.04	0.50	0.50	4.0	56	70	67	61	19.9	20.9	290	575	543	148	162	714	802	1.55	† 31	0.0085	Good
	1:5.3	1	2.65	2.65	5.2	0.02	0.51	0.51	4.0	61	81	79	72	16.5	20.8	1200	1725	1480	244	241	1317	1315	1.81	† 75	0.0120	Good
	1:3.7	1	1.85	1.85	7.6	0.01	0.52	0.52	4.0	64	93	89	79	13.6	19.2	2390	2765	2450	279	317	1639	1736	2.16	† 64	0.0125	Good
	1:2.9	1	1.45	1.45	9.5	0.01	0.51	0.51	6.0	68	97	92	84	11.3	16.7	2990	2978	2650	298	(189)	1390	1341	2.40	† 95	0.0125	Very Good
Sintered Fly Ash	1:9.2	1	4.60	4.60	3.5	0.05	0.59	0.59	5.0	60	87	85	79	15.8	22.3	400	775	795	171	194	1063	1138	2.19	† 54	0.0095	Poor
	1:6.0	1	3.00	3.00	5.1	0.05	0.58	0.58	5.0	61	92	87	82	15.2	21.2	1170	1800	1736	298	255	1453	1519	2.80	† 100	0.0100	Fair
	1:4.2	1	2.10	2.10	7.1	0.04	0.55	0.55	4.0	67	101	97	89	13.6	20.7	2620	3210	3130	395	414	1882	1935	2.88	† 100	0.0125	Fair
	1:3.1	1	1.50	1.50	9.4	0.03	0.60	0.60	6.0	73	107	101	93	11.6	19.0	3825	4200	3985	447	438	1979	2051	3.11	† 100	0.0095	Fair to Good
Pumice	1:9.2	1	1.85	7.35	2.9	0.04	0.20	0.80	3.0	63	74	64	57	29.6	29.6	960	1180	1210	105	118	480	610	1.73	77	0.0130	Poor to Fair
	1:5.3	1	1.06	4.24	4.7	0.03	0.18	0.74	4.0	73	81	70	63	25.5	27.6	1540	1765	1782	166	119	385	588	1.89	92	0.0170	Good
	1:3.7	1	0.74	2.96	6.6	0.02	0.18	0.78	6.0	74	85	76	67	22.5	25.6	2125	2045	2115	(121)	(125)	420	493	2.08	78	0.0165	Very Good
	1:2.7	1	0.54	2.16	8.1	0.02	0.13	0.68	5.0	79	89	79	73	19.0	24.1	2330	2170	2250	(65)	(132)	530	609	2.39	99	0.0150	Good
ESh	1:9.0	1	4.05	4.95	3.2	0.03	0.48	0.59	4.0	61	100	96	88	13.8	21.3	750	1385	1600	255	190	1817	1838	3.19	† 15	0.0090	Good
	1:5.9	1	2.65	3.25	4.9	0.02	0.49	0.58	5.0	58	106	104	95	10.6	17.5	2380	4000	4250	470	666	2479	2748	4.03	† 79	0.0115	Good
	1:4.1	1	1.84	2.26	6.8	0.02	0.46	0.43	4.0	61	108	106	97	8.7	14.7	4500	6205	6400	592	792	2807	2938	4.02	† 91	0.0090	Good
	1:3.0	1	1.35	1.65	8.6	0.02	0.43	0.37	4.0	64	111	109	100	8.4	14.6	3580	7045	7050	533	755	2646	3000	4.08	† 97	0.0100	Very Good
ESl	1:9.0	1	4.05	4.95	3.1	0.03	0.47	0.57	3.0	61	78	74	73	12.5	14.7	758	1380	1265	286	239	1095	1244	2.80	100	0.0113	Good
	1:5.2	1	2.34	2.86	5.0	0.03	0.42	0.55	7.0	59	81	79	74	7.5	9.5	1680	2300	2165	371	305	1255	1365	2.80	100	0.0103	Good
	1:3.7	1	1.67	2.03	6.9	0.03	0.42	0.55	5.0	65	87	83	87	4.7	6.6	2900	3350	3470	448	425	1505	1644	2.99	99	0.0093	Very Good
	1:2.8	1	1.28	1.57	8.7	0.03	0.42	0.51	5.0	66	93	91	85	5.3	7.6	3770	4005	4185	485	501	1740	1898	3.40	99	0.0093	Very Good
EC	1:9.0	1	3.60	3.40	3.1	0.03	0.41	0.61	4.0	46	86	79	75	16.4	20.6	623	1285	1369	254	217	837	1086	2.82	71	0.0120	Fair to Good
	1:6.0	1	2.10	3.90	4.4	0.03	0.34	0.64	3.0	46	89	85	78	10.3	13.8	1746	2596	2975	334	351	1060	1460	3.15	76	0.0123	Good
	1:3.8	1	1.38	2.47	6.7	0.03	0.33	0.61	3.0	50	95	90	84	7.0	10.0	3256	4184	4275	371	363	1175	1474	3.39	66	0.0120	Good
	1:2.4	1	0.84	1.56	9.8	0.03	0.30	0.56	3.0	57	102	100	90	4.2	6.6	3980	4740	5385	(256)	(199)	1473	1859	3.51	79	0.0095	Good to Very Good
Sand and Gravel	1:10.0	1	4.20	5.80	3.2	0	0.49	0.67	3.0	43	143	139	132	6.8	15.0	1040	1805	1706	344	299	2400	2861	6.75	24	0.0075	Fair to Good
	1:6.0	1	2.52	3.48	5.5	0	0.52	0.71	4.0	42	146	144	137	4.8	11.3	4040	4810	4710	611	488	3680	4376	9.57	86	0.0060	Good
	1:4.5	1	1.89	2.61	7.0	0	0.49	0.67	4.0	39	147	146	136	4.3	9.9	4540	5580	5140	611	675	3500	4373	7.94	83	0.0060	Very Good
	1:3.2	1	1.35	1.85	9.0	0	0.45	0.62	6.0	49	144	143	136	4.1	9.3	5190	6250	5780	536	(521)	3540	4172	6.20	60	0.0048	Very Good

* Based on weight of cement, * * Based on dry aggregate † Based on average of two specimens





Legend: ESh—expanded shale, EC—expanded clay, S1—expanded slag No. 1, S2—expanded slag No. 2, S3—expanded slag No. 3, ESl—expanded slate, D—sintered diatomite, P—perlite, V—exfoliated vermiculite

Fig. 5 (left)—Relation between compressive strength and cement content
Fig. 6 (right)—Relationship between compressive strength and specific weight

ally required more cement to develop comparable strengths. This, however, was not always true. For example the pumice concrete required approximately 3 bags of cement per cu yd, whereas expanded slag No. 1 concrete required between 5 and 7 bags to produce the same strength, notwithstanding the latter's 50 percent greater weight.

The transverse strengths, obtained from 4 x 4 x 20-in. prisms, of some of the concretes are plotted with respect to their compressive strengths in Fig. 7. Not all concretes were included in this comparison because some specimens developed shrinkage cracks during the drying period and consequently gave low transverse strength values. It is apparent from the data that were plotted that there was a consistent relation between these two properties.

Shrinkage

Data on drying shrinkage were obtained from three like specimens, each one representing a batch of concrete. Specimens were 3 x 4 x 12-in. prisms cured in a fog room for 7 days and dried at a constant temperature of about 70 F and a relative humidity of 55 percent.

The rate at which shrinkage progressed for each concrete from the time specimens were removed from the fog room until they were dry is shown in Fig. 8. Values of total shrinkage at the end of 100 days are listed in Table 4. With a few exceptions, the rate and total shrinkage were roughly the same for all concretes containing a given aggregate.

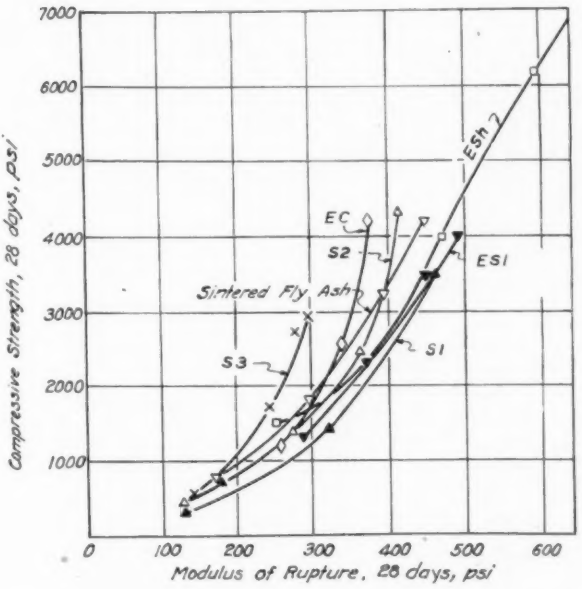


Fig. 7—Relationship between modulus of rupture and 28-day compressive strength

Legend: ESh—expanded shale, EC — expanded clay, S1 — expanded slag No. 1, S2—expanded slag No. 2, S3—expanded slag No. 3, ESl—expanded slate, D—sintered diatomite, P—perlite, V—exfoliated vermiculite

The differences that did occur were probably within the normal variation between samples and therefore not significant. There was no consistent relation between the cement content and shrinkage nor was there any consistent relation, except in a very general way, between shrinkage and the quantity of mixing water used per unit volume of concrete.

The highly absorptive aggregates, such as exfoliated vermiculite, sintered diatomite, and perlite produced concretes having relatively high shrinkage. This was evident not only from measurements but from the characteristic random cracking or crazing of many test specimens containing aggregates of this nature. Two examples of rather badly crazed specimens are shown in Fig. 9.

A comparison of the average shrinkage of each concrete at the end of 100 days is shown in Fig. 10. The concretes containing the expanded clay, shale, slate, and sintered fly ash aggregates as well as those containing the expanded slags, with the possible exception of expanded slag No. 1, were essentially alike in shrinkage properties. Their maximum shrinkage was roughly twice that of sand-gravel concrete.

Resistance to freezing and thawing

Freezing-and-thawing tests were made on 3 x 4 x 12-in. prisms, cured in the same manner as the strength test specimens. Freezing was done in air at −50 F for a period of 4 hours and thawing was accomplished by soaking specimens in water at approximately 60 F for a period of about 15 hours. Progressive deterioration of the concretes was

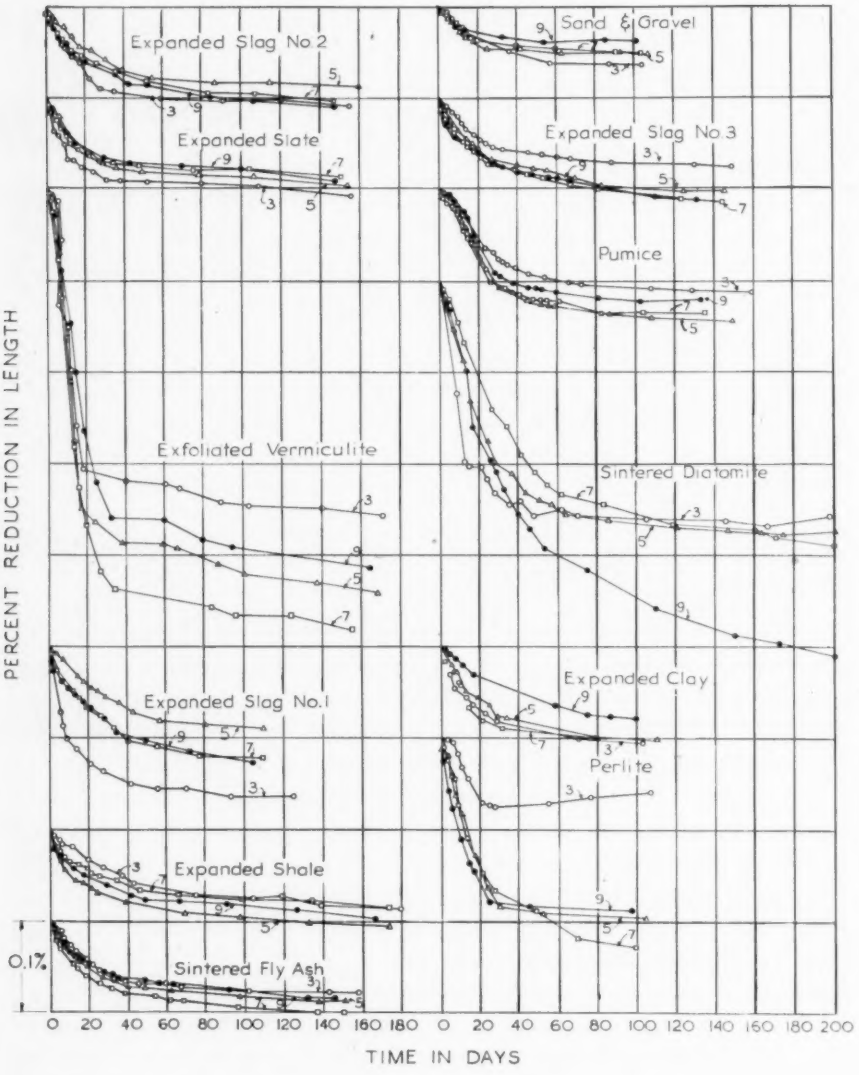


Fig. 8—Shrinkage curves for various concretes

ascertained by observing the change in dynamic elastic modulus and by visual inspection to detect surface disintegration. The performance of the various concretes was evaluated by a method based on Stanton Walker's *durability factor*.[1] *Durability factors* for each of the concretes are listed in Table 4.

[1]Walker, Stanton, "Freezing and Thawing Tests of Concrete Made With Different Aggregates," ACI JOURNAL, June 1944, *Proc.*, V. 40, p. 575.

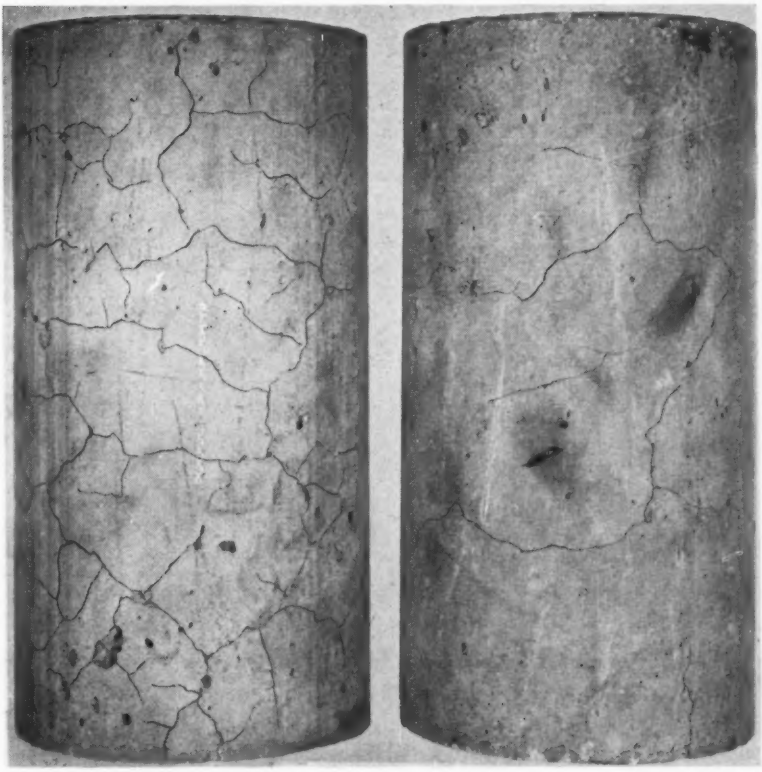


Fig. 9—Examples of badly crazed specimens

Typical curves showing the reduction in dynamic E due to freezing and thawing are shown in Fig. 11, and examples of particularly severe surface disintegration are shown in Fig. 12. The type of aggregate in these specimens appeared to be especially vulnerable to the freezing tests since it required only 3 cycles of freezing and thawing to cause the deterioration shown at the right of the photograph. The specimens to the left are 9, 7, 5 and 3-bag concretes, respectively, as they appeared before testing.

Generally, the richer the mix the more resistant the concrete was to laboratory freezing and thawing although there were other factors that affected this property. One exception which involved a rich mix was the 9-bag expanded slag No. 1 which failed earlier than the 3-bag mix of the same material. The 9-bag mix did not contain an air-entraining agent implying that air entrainment considerably improved the resistance of lightweight aggregate concretes to freezing and thawing.

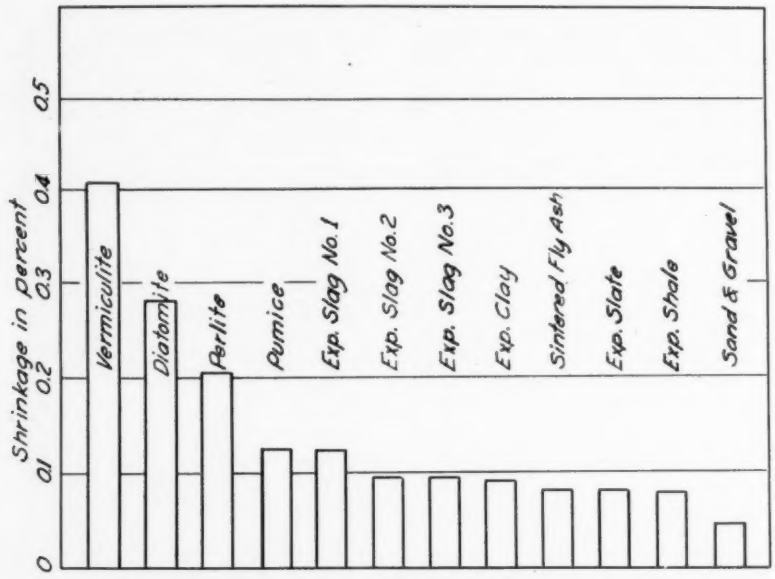
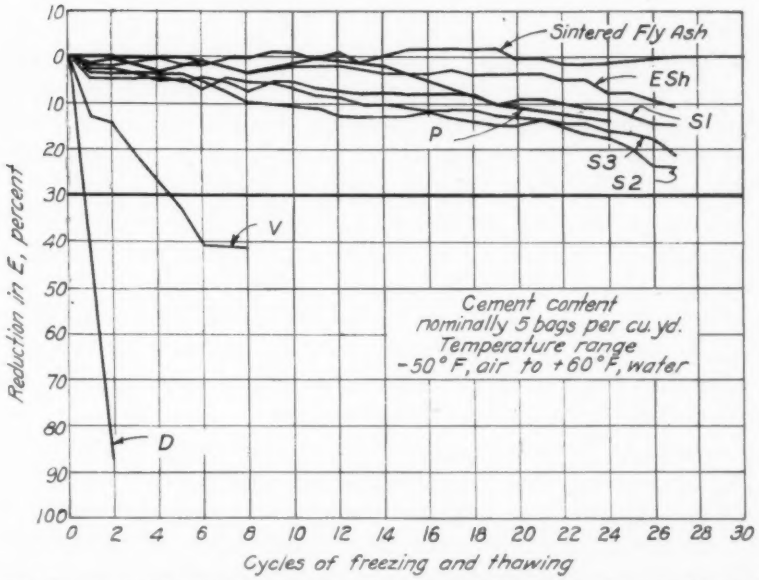


Fig. 10—Average drying shrinkage in 100 days

Since water in the pore structure is fundamentally responsible for the deterioration of concrete subjected to freezing temperatures, it is reasonable to expect some relationship, other things being equal, between the absorption of the concrete and its resistance to freezing and thawing. In Fig. 13, the durability factors for the 5, 7 and 9-bag concrete mixes containing air-entraining agents were plotted with respect to their volumetric absorption. In the region of the lower absorptions, where most of the data were obtained, there is a considerable scattering of values. However, considering all of the data, the trend is evident. In this connection it should be noted that the air content of the various concretes which, as shown above, apparently had a marked influence on their resistance to freezing and thawing, was not systematically controlled; consequently, such a scattering of values could be expected.

The 3-bag mixes were not included in the graph because, in general, their resistance to freezing and thawing was inferior to the other concretes regardless of their absorption.

It should be pointed out that correlation between laboratory freezing-and-thawing tests and the durability of concrete exposed to freezing temperatures in service is not definite. Laboratory tests do, however, serve to indicate the probable performance of the concrete in service. Therefore, very poor laboratory performance would indicate the possibility of poor service in the field.



Legend: ESh—expanded shale, EC—expanded clay, S1—expanded slag No. 1, S2—expanded slag No. 2, S3—expanded slag No. 3, ESl—expanded slate, D—sintered diatomite, P—perlite, V—exfoliated vermiculite

Fig. 11—Typical curves showing reduction in dynamic E due to freezing and thawing

Fig. 12—Examples of severe surface disintegration

Elastic properties

The modulus of elasticity of each concrete was determined by both static and sonic or dynamic methods from 6 x 12-in cylinders at an age of 28 days. Both methods were applied to each test specimen. Inasmuch as the dynamic E corresponds to the initial elastic modulus, this value was also determined from the static test.

Values of E obtained by the two methods are compared in Table 4. About one-half of the values agree within 10 percent and three-quarters of the values agree within 25 percent. Considering that the personal factor enters into the determination of the initial tangent modulus by the static method and it was therefore subject to some variation, agreement between the two methods is reasonably good. The sonic values, however, were higher in almost all instances.

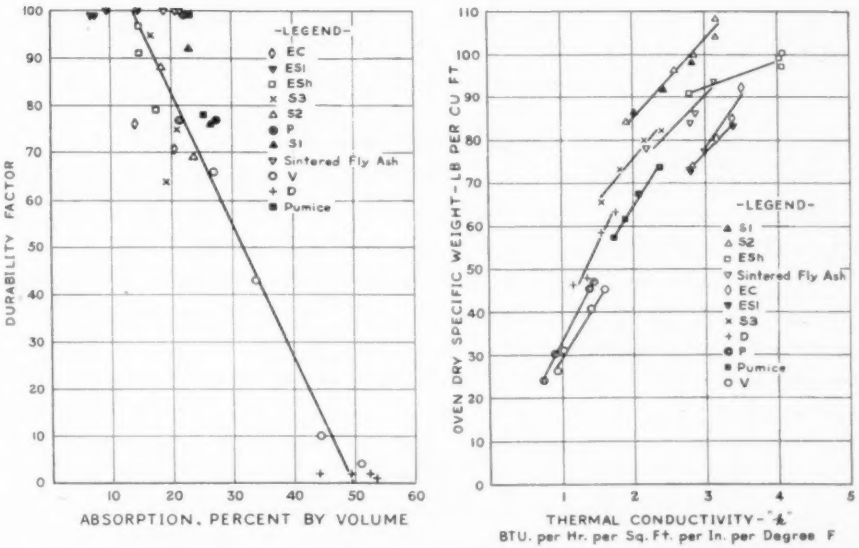
As expected, the elastic modulus was influenced to a great extent by the type of aggregate and usually by the cement content. The concretes containing the softer aggregates had very low values which were increased little, if any, by an increase in cement. On the other hand, concretes containing the harder materials such as expanded clay, shale, slate and slags developed values between 800,000 and 3,000,000 psi and, in general, the effect of cement content was very marked.

Thermal conductivity

The thermal conductivities of each concrete were determined by means of the guarded hot-plate test of 8 x 8 x 1-in thick oven-dried samples. The tests were conducted with the warm side of the specimen at 140 F and the cold side at 98 F.

Thermal conductivity or k values representing heat loss in BTU per hour per sq ft of area per degree F temperature difference per in. of thickness are given in Table 4. In general, the heavier the concrete, the greater the conductivity. However, concretes made with certain aggregates showed better insulating properties for a given weight than others. Fig. 14 indicates the relation between the weight and the thermal conductivity for the various lightweight concretes. It is interesting to note that two of the slag concretes, expanded slags No. 1 and No. 2 showed almost identical insulating properties. Although this may have been merely a coincidence, it may be pointed out that these two slags were similar in appearance and general properties and both were produced by the open-pit process, whereas expanded slag No. 3 was lighter in weight and color and was produced by a machine process.

Within the range of data, the relation between the weight of the concretes and their thermal conductivity appeared to be approximately linear for each type of aggregate. It should be noted that the k values were obtained on oven-dried specimens of concrete. There is evidence,



Legend: ESh—expanded shale, EC—expanded clay, S1—expanded slag No. 1, S2—expanded slag No. 2, S3—expanded slag No. 3, ESI—expanded slate, D—sintered diatomite, P—perlite, V—exfoliated vermiculite

Fig. 13 (left)—Relationship of Durability Factor to volumetric absorption
Fig. 14 (right)—Relationship between oven-dry weight and thermal conductivity

some published[2], to indicate that small percentages of moisture in the concretes could affect these values considerably.

SUMMARY

The results of the study may be briefly summarized as follows:

1. Most of the lightweight aggregates, because of the angular character of the individual particles, coupled with their peculiar absorptive properties, required techniques for testing, proportioning and mixing differing somewhat from those for ordinary natural dense aggregate mixtures.

2. Concretes, weighing as little as one-fifth the weight of ordinary concrete, were made. They developed little strength but exhibited a thermal conductivity as low as one-tenth that of sand-gravel concrete. Other lightweight aggregate concretes developed compressive strengths between 1000 and 7000 psi, and weighed approximately one third to two-thirds as much as sand-gravel concrete. Their thermal conductivities when dry were about one-fourth to one-half those of ordinary dense concrete.

3. Most lightweight aggregates had relatively high water absorption and shrinkage of the concretes made with them was approximately two or more times that of dense aggregate concrete.

[2]Tyner, Mack, "Effect of Moisture on Thermal Conductivity of Limerock Concrete," ACI JOURNAL, Sept. 1946, *Proc.* V. 43, p. 9.

4. Except for lean 3-bag mixes, lightweight concrete containing air-entraining agents, in general, exhibited remarkable resistance to rather severe laboratory freezing-and-thawing tests.

ACKNOWLEDGMENTS

The authors wish to acknowledge the valuable assistance of the following staff members of the National Bureau of Standards for the portions of the test data which they contributed to the study.

H. E. Robinson and T. W. Watson tested the concretes for thermal conductivity; R. A. Clevenger and V. T. Tyler conducted all of the aggregate tests and Rudolph Valore, Jr. gave valuable advice and suggestions concerning the freezing-and-thawing tests and their interpretation.

APPENDIX

The quantities of materials per cu yd of concrete given in Table 4 are based on the measured yield of concrete. Inasmuch as many lightweight aggregates vary considerably in absorption, specific gravity, and unit weight, the calculated quantities based on these properties are not entirely consistent with the values given in Table 4. Ordinarily, the differences are not of practical significance. However, in order that there be no confusion for the reader making a careful study of Tables 2 and 4, Table 5 is included with revised values of proportions which are consistent with the properties listed in Table 2.

TABLE 5—CALCULATED QUANTITIES OF MATERIALS IN LIGHTWEIGHT-AGGREGATE AND SAND-GRAVEL CONCRETES

Aggregate	Bags of cement per cu yd	Cu yd of agg. per cu yd of conc.		Water gal/cu yd	Aggregate	Bags of cement per cu yd	Cu yd of agg. per cu yd of conc.		Water gal/cu yd
		fine	coarse				fine	coarse	
Vermiculite	2.7	0.97		99	Sintered fly ash	3.4	0.56	0.56	57
	4.5	0.88		95		5 0	0.56	0.56	60
	6.1	0.88		93		7.1	0.55	0.55	67
	7.6	0 84		91		9.2	0.53	0.53	72
Diatomite	2.7	0 91		114	Pumice	3.3	0.20	0.81	71
	4.1	0 90		112		4.8	0.19	0.75	74
	6.3	0.86		117		6.5	0.18	0.71	72
	8.8	0.78		118		8.2	0.16	0.65	74
Perlite	2.4	0 96		83	Expanded shale	3.1	0.52	0 52	60
	4.1	0.84		70		4.8	0 47	0.61	58
	7.3	0.92		67		6.7	0.44	0.57	59
	8.1	0.87		69		8.4	0.42	0.53	62
Slag No. 1	3.3	0.93		79	Expanded slate	3.0	0 45	0.52	44
	4.8	0.88		76		4 9	0 42	0.51	44
	6.6	0.86		72		6.6	0.41	0.48	48
	9.3	0.92		84		8.2	0.40	0.46	53
Slag No. 2	3.2	0.65	0.37	57	Expanded clay	2.9	0 39	0 57	59
	4.5	0.74	0.39	63		4.3	0.34	0.63	58
	6.4	0.73	0.38	68		6.6	0.32	0.60	63
	9.4	0.67	0.36	71		9.6	0.30	0 55	65
Slag No. 3	2.9	0.48	0.48	54	Sand-gravel	3.1	0.49	0.65	42
	5.1	0.50	0.50	59		5.4	0 45	0.63	42
	7.6	0.52	0.52	63		6.9	0.46	0.60	46
	9 2	0.49	0.49	66		8.8	0.32	0 66	47

Relative changes in elevation of foundations may endanger the stability of a structure. Mr. Leonards proposes a method of analysis which reduces the computations necessary to estimate the effects of settlement in large frames.

Analysis of Building Frames with Unsymmetrical Differential Settlement of the Foundations*

By G. A. LEONARDS†

SYNOPSIS

The rational design of building frames founded on soil requires a consideration of the effect of settlements caused by the building loads. This paper discusses a procedure for estimating the stresses in such frames due to unsymmetrical differential settlement of the foundations. The procedure is based on the slope-deflection method for analyzing rigid frames, but adapts it to permit a relatively rapid solution by the use of successive approximations.

INTRODUCTION

All building foundations, except those carried down to solid rock, settle appreciably during the life of the structure. Experience and theoretical investigations have shown that the stresses imposed by relatively small differential settements can endanger the stability of a structure. Notwithstanding this fact, building frames are designed, in general, on the assumption that the footings suffer no relative change in elevation. There are a number of reasons for this anomaly, the most important of which are:

1. The uncertainties involved in determining the amount, and particularly, the time-rate of settlement of the foundations.

2. The uncertainty regarding the amount of stress relief that is caused by "creep" of the materials comprising the building frame; since, in many cases, settlements will take place over long periods.

3. The tedious and time-consuming computations required to estimate the effect of the settlements (assuming that the frame as a whole behaves elastically) particularly in large frames subjected to unsymmetrical differential settlements.

*Received by the Institute Feb. 15, 1949. Title No. 45-38 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 9, May 1949, *Proceedings* V. 45. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Sept. 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

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While, admittedly, there is a greater need for clarification of items one and two, this paper deals exclusively with item three, assuming certain arbitrary differential settlements.

DEVELOPMENT OF FUNDAMENTAL EQUATIONS

In building frames, if the members as a whole behave elastically (obey Hooke's law), the deformations are small and those due to shear and axial stresses are neglected; and if the members are of constant cross section, then the moments at the ends of any member in the frame can be expressed by the slope-deflection equation* as follows:

$$M_{AB} = M_{FAB} + \frac{2EI}{L}\left[-2\Theta_A - \Theta_B + 3\frac{\Delta}{L}\right] \quad \ldots\ldots\ldots\ldots (1)$$

where, M_{AB} = the moment at A in the member AB
M_{FAB} = the fixed end moment at A due to loads on the member AB
E = the modulus of elasticity of the member
I = the moment of inertia of the member
L = the length of the member
Θ_A = the rotation of end A
Θ_B = the rotation of end B
Δ = the relative displacement of end A from end B

Letting $\frac{2EI}{L} = K_{AB}$ and $\frac{\Delta}{L} = R_{AB}$, Eq. (1) becomes

$$M_{AB} = M_{FAB} + K_{AB}\left[-2\Theta_A - \Theta_B + 3R_{AB}\right] \quad \ldots\ldots\ldots\ldots (2)$$

Consider the portion of a building frame shown in Fig. 1. The columns are designated by the letters a, b, c, etc., and the floor levels by the numbers 1, 2, 3, etc. A joint is designated by using the above notation as a coordinate system. In Fig. 1 the circled joint is designated $q6$. Girders and columns are referred to by first naming the joint and then the letter or number at the far end of the member. Girder to the right of the circled joint in Fig. 1 is designated $q6,r$. Stiffnesses $\left(K = \frac{2EI}{L}\right)$ also are designated in this manner.

In general, differential settlements result in relative displacements of the ends of the members framing into a particular joint. Considering the relative displacements shown in Fig. 1, the moments at the ends of the members that frame into joint $q6$ are (from Eq. 2):

*Parcel, John I., and Maney, George A., "Statically Indeterminate Stresses," 2nd Edition, John Wiley and Sons, Inc., 1947

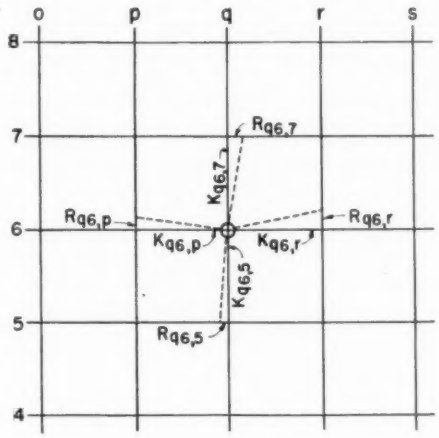


Fig .1—Portion of a building frame showing coordinate system of designating levels and members

$$M_{q6,r} = M_{Fq6,r} + K_{q6,r}\left[-2\Theta_{q6} - \Theta_{r6} + 3R_{q3,r}\right]$$
$$M_{q6,p} = M_{Fq6,p} + K_{q6,p}\left[-2\Theta_{q6} - \Theta_{p6} + 3R_{q6,p}\right]$$
$$M_{q6,7} = M_{Fq6,7} + K_{q6,7}\left[-2\Theta_{q6} - \Theta_{q7} + 3R_{q6,7}\right]$$
$$M_{q6,5} = M_{Fq6,5} + K_{q6,5}\left[-2\Theta_{q6} - \Theta_{q5} + 3R_{q6,5}\right]$$

Adding, and since the sum of the moments around any joint equals zero

$$0 = \Sigma M_{Fq6} - 2\Theta_{q6}\Sigma K_{q6} - \Sigma K_i\Theta_i + 3\Sigma K_C R_C + 3\Sigma K_G R_G$$

where, Θ_{q6} = the rotation of joint $q6$

ΣM_{Fq6} = the summation of the fixed end moments at joint $q6$

$\Sigma K_G R_G$ = the summation of the product of the stiffness of each girder framing into joint $q6$ by the corresponding R (known or estimated) due to the relative movement of the girders left and right of $q6$.*

$\Sigma K_C R_C$ = the summation of the product of the stiffness of each column framing into joint $q6$ by the corresponding R (unknown) due to sidesway.

$\Sigma K_i\Theta_i$ = the summation of the product of the stiffness of each member framing into joint $q6$ by the corresponding rotation of the joints at the far end of each member.

ΣK_{q6} = the summation of the stiffnesses of all the members framing into joint $q6$.

Or, an exact equation for the rotation of any joint in the frame is

$$\Theta_{q6} = \frac{\Sigma M_{Fq6} + 3\Sigma K_G R_G + \Sigma K_C R_C - \Sigma K_i\Theta_i}{2\Sigma K_{q6}} \quad \ldots\ldots\ldots\ldots (3)$$

The sum of the moments top and bottom of any column, say $q6,5$, is

$$M_{q6,5} + M_{q5,6} = K_{q6,5}\left[-2\Theta_{q6} - \Theta_{q5} + 3R_{q6,5}\right] + K_{q5,6}\left[-2\Theta_{q5} - \Theta_{q6} + 3R_{q5,6}\right]$$

*In the example to follow, these relative displacements for all the upper stories are assumed to equal the differential settlements of the footings; that is, the axial deformation of the columns will be neglected.

$$= K_{q6,5}\,[-3\Theta_{q6} - 3\Theta_{q5} + 6R_{q6,5}] \quad \text{.....(4a)}$$

Since $K_{q6,5} = K_{q5,6}$ and $R_{q6,5} = R_{q5,6}$

The sum of the moments top and bottom of all the columns in a given story, say 56, is the sum of the terms given in Eq. (4a) for all the columns in the story. Using the symbol n to represent the column rows a, b, c, etc., in a given story, this sum becomes

$$\Sigma\{M_{n6,5} + M_{n5,6}\} = \Sigma\{K_{n5,6}[-3\Theta_{n5} - 3\Theta_{n6} + 6R_{n5,6}]\} \quad \text{....(4b)}$$

If the total external shear is zero (vertical loads only on the frame), the sum of the moments top and bottom of all the columns in a given story equals zero, then from Eq. (4b)

$$6\Sigma K_{n5,6} \cdot R_{n5,6} - 3\Sigma K_{n5,6} \cdot \Theta_{n5} - 3\Sigma K_{n5,6} \cdot \Theta_{n6} = 0 \quad \text{.....(4c)}$$

The sidesway in a given story is substantially constant; hence, if all the columns are the *same height*, then, from Eq. (4c), an exact equation for R of the columns due to sidesway in the story 56 is

$$R_{56} = \frac{\Sigma\{K_{n5,6}[\Theta_{n5} + \Theta_{n6}]\}}{2\Sigma K_{n5,6}} \quad \text{.....(4)}$$

where, R_{56} = R due to sidesway in the story 56 (R_C in Eq. (3))

$\Sigma\{K_{n5,6}[\Theta_{n5} + \Theta_{n6}]\}$ = the summation of the product of the stiffness of each column in the story 56 by the sum of the corresponding rotations of the joints at the top and bottom of each column.

$\Sigma K_{n5,6}$ = the summation of the stiffnesses of all the columns in the story 56.

SOLUTION OF EQ. (3) AND (4) BY SUCCESSIVE APPROXIMATIONS

1. Compute the fixed end moments. Releasing *alternate* joints and temporarily neglecting sidesway, the rotation of these alternate joints is computed, which, from Eq. (3) equals

$$\frac{\Sigma M_F + 3\Sigma K_G R_G}{2\Sigma K}$$

2. Still neglecting sidesway, release the remainder of the joints and compute the corresponding rotations of these joints, which from Eq. (3) equals

$$\frac{\Sigma M_F + 3\Sigma K_G R_G - \Sigma K_i \Theta_i}{2\Sigma K}$$

3. Adjust the rotations computed in step one for the rotations of the alternate joints computed in step two using $-\frac{\Sigma K_i \Theta_i}{2\Sigma K}$. The rotations computed in step two can now be adjusted for the corrections to the

rotations of the alternate joints just computed, but this adjustment is, generally, so small that it need not be carried out.

4. Using the rotations determined by the above steps, compute the first value of R_C due to sidesway for each story, using Eq. (4).

5. Using $\frac{3\Sigma K_C R_C}{2\Sigma K}$ from Eq. (3), compute the rotations caused by the sidesways determined in step four. If these rotations are large, compute the *additional* R_C that they cause. Compute the rotations due to the additional R_C.

6. Distribute the additional rotations computed in step five using $-\frac{\Sigma K_i \Theta_i}{2\Sigma K}$.*

ILLUSTRATIVE EXAMPLE

Computations for moments in a three-bay, four-story, reinforced concrete frame caused by arbitrarily assigned differential settlements of the footings are shown in Fig. 2. In this example, clockwise joint rotations (Θ) and member rotations $\left(\frac{\Delta}{L} = R\right)$ are called positive; accordingly, *end moments* that cause clockwise rotations of the *joint* are called positive. Sample computations for the joint $b4$ are as follows:

1. The first value of the joint rotation equals

$$\Theta^{\mathrm{I}} = \frac{3\Sigma K_G R_G}{2\Sigma K} = \frac{3[2.02 \times 43.8 - 1.74 \times 18.9]}{9.12} = +18.3$$

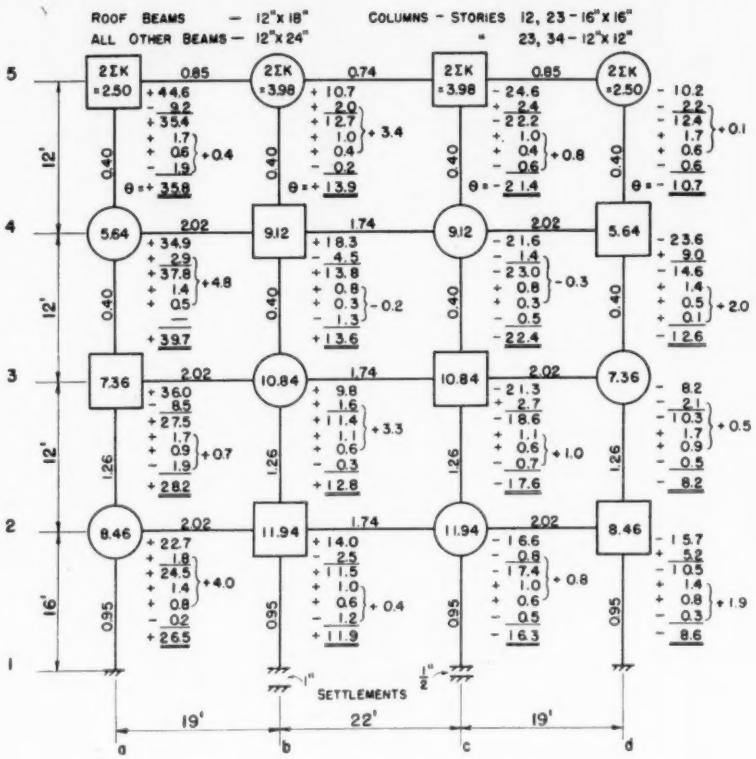
3. Omitting step two, the adjustment for the rotations of the adjacent joints equals

$$\Theta^{\mathrm{II}} = -\frac{\Sigma K_i \Theta_i}{2\Sigma K}$$

$$= \frac{-2.02 \times 34.9 - 0.40 \times 10.7 + 1.74 \times 21.6 - 0.40 \times 9.8}{9.12} = -4.5$$

*In most cases, solutions of sufficient accuracy for design purposes can be obtained as follows:

1. Carry out steps one and two as previously indicated.
2. Adjust the rotations computed in step one for the rotations of the alternate joints computed in step two using $-\frac{\Sigma K_i \Theta_i}{2\Sigma K}$.
3. Using the rotations determined by the above steps, compute the value of R_s due to sidesway for each story, using Eq. (4).
4. Using $\frac{3\Sigma K_s R_s}{2\Sigma K}$ from Eq. (3), compute the rotations caused by the sidesways computed in step three. Distribute these additional rotations using $-\frac{\Sigma K_i \Theta_i}{2\Sigma K}$.



COMPUTATION OF RELATIVE STIFFNESS

MEMBER	$I = \frac{bd^3}{12}$ in.⁴	L in.	$K = \frac{2EI}{L}$ * kip ft.	RELATIVE K †
COLS. -12"x12"	1728	144	4000	0.40
" -16"x16"	5460	144	12630	1.26
" -16"x16"	5460	192	9470	0.95
GIRDERS -12"x18"	5832	228	8530	0.85
" - "	5832	264	7360	0.74
" -12"x24"	13824	228	20200	2.02
" - "	13824	264	17420	1.74

* $E = 2 \times 10^6$ p.s.i. † MULTIPLYING FACTOR = 10^{-4}

MEMBER ROTATIONS DUE TO SETTLEMENT

GIRDERS	SETTLEMENT Δ — in.	L in.	$R_0 = \frac{\Delta}{L} \times 10^4$
ab	1.0	228	+43.8
bc	0.5	264	-18.9
cd	0.5	228	-21.9

MEMBER ROTATIONS DUE TO SIDESWAY

STORY	FINAL R_c
12	+1.70
23	+3.61
34	+4.20
45	+4.50

Fig. 2—Analysis of a reinforced concrete building frame

4. The first value of the sidesway in story 45 equals

$$R^{\mathrm{I}}_{C,45} = \frac{\Sigma K_C[\Theta_{b4} + \Theta_{b5}]}{2\Sigma K_C}$$

$$= \frac{0.4[+35.4 + 12.7 - 22.2 - 12.4 + 37.8 + 13.8 - 23.0 - 14.6]}{2[4 \times 0.40]}$$

$$= +3.44$$

5. The corresponding rotation due to this sidesway equals

$$\Theta^{\mathrm{III}} = \frac{3\Sigma K_C R_C}{2\Sigma K} = \frac{3[+3.44 \times 0.40 + 3.00 \times 0.40]}{9.12} = +0.8$$

The adjustment to the sidesway due to the rotations just computed equals

$$R^{\mathrm{II}}_{C,45} = \frac{0.40[+1.40 + 0.80 + 0.80 + 1.40 + 1.70 + 1.00 + 1.00 + 1.70]}{2[4 \times 0.40]}$$

$$= +1.22$$

The rotation due to this adjustment of the sidesway equals

$$\Theta^{\mathrm{IV}} = \frac{3[1.22 \times 0.40 + 1.25 \times 0.40]}{9.12} = +0.3$$

6. The distribution of the additional rotations computed in step five equals

$$\Theta^{\mathrm{V}} = \frac{-0.40 \times 3.4 + 1.74 \times 0.3 - 0.40 \times 3.3 - 2.02 \times 4.8}{9.12} = -1.3$$

The moments at the ends of the members framing into joint $b4$ equal:

Moment	$= K[-2\Theta_A - \Theta_B + 3R]$		
$b4,c$	$= 1.74[-27.2 + 22.4 - 57.6]$	$= -107.0$	
$b4,a$	$= 2.02[-27.2 - 39.7 + 131.4]$	$=$	$+130.2$
$b4,5$	$= 0.40[-27.2 - 13.9 + 13.5]$	$= -11.0$	
$b4,3$	$= 0.40[-27.2 - 12.8 + 12.6]$	$= -11.0$	
		$-129.0^{k'}$	$+130.2^{k'}$

ADVANTAGES

In the author's opinion, the method outlined has the following advantages:

1. It converges very rapidly. In the illustrative example, computation of the rotations due to the second value of sidesway and the distribution of these rotations could have been omitted, for practical purposes.

2. Only one series of computations are required for each joint cutting down the amount of "book-keeping" considerably; especially for large frames.

A minor disadvantage of the method occurs when the columns in a given story are not all of the same height, in which case Eq. (4) becomes much more complex.

ACKNOWLEDGMENTS

The method of analysis outlined in this paper is, essentially, an adaptation of the Maney-Goldberg* analysis of rigid frames by the method of alternate joints. Special thanks are due L. T. Wyly, Professor and Head, Structural Engineering Department, Purdue Univ., for his advice and guidance, and to R. I. Kaufman, formerly Instructor at Purdue Univ., for checking the computations.

*Maney, G. A., and Goldberg, J. E., "Simplified Methods for the Analysis of Multiple Joint Rigid Frames", *Bulletin* No. 7, V. XXXIII, Northwestern Univ., Chicago, Ill.

The authors report further findings in a study of factors affecting air entrainment. Earlier results of this investigation were published in the November 1948, and February 1949 Journals.

Effects of Mixing Time, Size of Batch and Brand of Cement on Air Entrainment*

By E. W. SCRIPTURE, Jr.,† and F. J. LITWINOWICZ‡

SYNOPSIS

A program, partially described in previous papers, to investigate various factors affecting the amount of air entrained in concrete has been continued. The factors covered in this paper are mixing time, size of batch, and brand of cement. The entrained air rises to a maximum in the early stages of mixing and thereafter decreases. Provided mixing is adequate, the size of batch does not appear to affect the amount of air entrained. Wide variations are found in the amounts of air entrained by different cements and it does not appear possible to correlate these variations with any easily and quickly determinable characteristic of the cement.

INTRODUCTION

In a previous paper§ the results of a study of the effects of slump, cement factor and sand-total aggregate ratio on air entrainment were reported. In a still earlier paper¶ an investigation of the influence of the size grading of the sand on air entrainment was described. It has not been the purpose of this work to attempt to evaluate specific air-entraining agents, the proportions in which they are to be used to produce a given effect or their manner of use, but holding these factors constant to evaluate one by one, under controlled conditions, other variable factors known to affect or suspected of affecting the quantity of air entrained in concrete mixes. Comparisons have been made with the three air-entraining agents now in most common use, in proportions

*Received by the Institute Feb. 10, 1949. Title No. 45-39 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No 9, May 1949, *Proceedings* V. 45. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Sept. 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

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§Scripture, E. W., Jr., and Litwinowicz, F. J., "Some Factors Affecting Air Entrainment," ACI JOURNAL, Feb. 1949, *Proc.* V. 45, p. 433.

¶Scripture, E. W., Jr., Hornibrook, F. B., and Bryant, D. E., "Influence of Size Grading of Sand on Air Entrainment," ACI JOURNAL, Nov. 1948, *Proc.* V. 45, p. 217.

commonly used in practice, and added at the mixer to avoid any complications which might be involved in intergrinding. The effects of mixing time, size of batch and brand of cement are reported here.

TEST PROGRAM

In general, four series of tests were made for each factor studied, one series of mixes with no air-entraining agent added and one series with each of three different air-entraining agents; Admixture A—a petroleum hydrocarbon insoluble fraction of a coal-tar hydrocarbon extract of pine wood, Admixture B—a triethanolamine salt of a sulfonated hydrocarbon, and Admixture C—a calcium lignosulfonate cement dispersing agent. In the investigation of each variable factor two sets of tests were made, one with a nominal 6-sack mix to represent fairly rich mixes, and one with a nominal 4½-sack mix to represent moderately lean mixes. Slumps were held to 3 to 5 in. but in any single series were kept within a 1-in. range. Sand was a local bank sand and the coarse aggregate was a crushed limestone. The size gradings and specific gravities of these aggregates are given in Table 1. The amounts of Admixtures A and B, 0.01 and 0.015 percent by weight of the cement, respectively, were selected to give between 4 and 5 percent of air in an average mix under normal conditions. Admixture C was used in accordance with the manufacturer's directions in the proportion of 0.5 lb. per sack of cement.

In the investigation of the effect of mixing time on entrained air time was varied from 2 to 60 minutes. In the field tests made with a nominal cement factor of 6 sacks per cu yd time varied from 15 to 90 minutes. In the investigation on size of batch a 2½-cu ft mixer was used and the batches varied from ½ to 2½ cu ft by ½ cu ft steps. The

TABLE 1—SIZE GRADINGS AND SPECIFIC GRAVITIES OF AGGREGATES

	Percent retained						
	Fine aggregate				Coarse aggregate		
Sieve size	Mixing time series	Batch size series	Cement brand series	Sieve size	Mixing time series	Batch size series	Cement brand series
No. 4	3	2	2	1½-in. screen	0	0	0
No. 8	11	11	11	1 -in. screen	58	44	42
No. 16	16	16	14	¾-in screen	33	37	37
No. 30	19	22	18	½-in. screen	5	15	16
No. 50	39	37	30	⅜-in. screen	1	3	3
No. 100	11	11	14	¼-in. screen	1	1	1
Pan	1	1	1	Pan	2	1	1
Sp. gr.	2.60	2.60	2.60	Sp. Gr.	2.76	2.80	2.75

same brand of cement, a normal portland cement Type I, was used throughout these two investigations.

For the study of the effect of brand of cement twelve cements from different sections of the country were used—nine were Type I normal portland cements, two were Type III, and one was a Type II cement of the same brand as one of the Type I cements. Surface areas and entrained air by the A.S.T.M. mortar test were determined. Although the latter test was not designed for use with non-air-entraining cements it was thought that there might be some relation between these characteristics of the cements and the air entrained in concrete mixes.

TEST PROCEDURE

The concrete mixes* were made in batches of approximately 2 cu ft, mixed 3 minutes in a 2½-cu ft tilting-drum concrete mixer, except, of course, that size of batch or time of mixing was varied in studies relating to them as factors. Slumps were measured and 6 x 12-in. cylinders were made, cured and broken to determine compressive strengths using appropriate A.S.T.M. standard methods. Air contents were calculated gravimetrically and determined immediately after mixing by the pressure method with a Klein-Walker air meter. Determinations of entrained air in standard mortar mixes were made in accordance with A.S.T.M. Method Designation C 185-47T. Determinations of surface area were made with the Blaine air permeability apparatus and converted by factor to Wagner values. Both values are given in Table 2.

TABLE 2—TYPES OF CEMENT, SURFACE AREAS AND AIR CONTENTS IN A.S.T.M. MORTAR TEST

Cement designation	Type	Percent air A.S.T.M. mortar test	Surface area, sq cm per gram	
			Blaine	Wagner
A	I	5.9	3000	1630
B	I	6.3	2900	1580
C	I	4.5	3070	1670
D	II	4.0	3290	1790
E	III	5.4	4070	2210
F	III	7.6	4770	2590
G	I	7.2	2700	1470
H	I	4.5	2800	1520
I	I	6.3	2830	1540
J	I	7.6	2980	1620
K	I	5.9	3040	1650
L	I	6.8	3110	1690

*Mix data on the test series reported in this paper are on file in the ACI Secretary's office and available at cost of reproduction.

TABLE 3—AIR ENTRAINED IN RELATION TO MIXING TIME —LABORATORY SERIES

Mixing time-min.	Air contents, percent			
	No addition	Admixture		
		A	B	C
Nominal 4½ sacks per cu yd				
2	1.8	5.3	6.0	3.8
5	1.1	5.2	5.5	3.4
10	1.0	4.2	4.9	2.1
15	1.0	3.3	4.2	1.9
30	1.0	2.5	3.4	1.1
60	1.0	2.0	1.5	1.0
Nominal 6 sacks per cu yd				
2	1.3	4.5	4.4	3.1
5	1.0	5.2	5.9	2.1
10	0.9	5.8	5.9	1.4
15	0.8	5.7	5.8	1.3
30	0.8	4.8	5.5	1.2
60	0.8	4.0	4.1	1.0

TEST RESULTS

Varying mixing time

Results of the air determinations with varying mixing time in the laboratory series are given in Table 3, the results from the field tests in Table 4. The laboratory tests differed from those usually made to investigate the effect of mixing time on the properties of concrete in that air determinations were made on separate batches of varying water content, using sufficient water in each mix to give approximately the same slump regardless of mixing time, whereas the usual practice is to use a fixed water-cement ratio and to allow the slump to vary, making successive air determinations on a single batch. There is, therefore, a steady increase in water-cement ratio with mixing time in all cases. On the other hand, in the field series a single large batch was made for each mix and test samples were taken from the same batch at the different mixing times.

In the nominal 4½-sack series, in all cases the air content was maximum at the shortest mixing time and thereafter decreased. In mixes with no addition the air content dropped rapidly from 1.8 percent at 2 minutes to 1.1 percent at 5 minutes mixing time. There was then a small drop between 5 and 10 minutes to 1.0 percent, and thereafter it remained constant. With Admixture C air was lost more slowly, but there was a regular reduction in air content from 3.8 percent after 2 minutes of mixing to 1.0 percent at 60 minutes. Admixtures A and B behaved in a similar manner but, starting at higher air contents of 5.3

TABLE 4—VARIATION OF AIR ENTRAINMENT AND SLUMP WITH MIXING TIME—FIELD SERIES

Mixing time, min.	No addition		Admixture					
			A		B		C	
	Percent air	Slump, in.	Percent air	Slump, in.	Percent air	Slump, in.	Percent air	Slump, in.
15	1.0	6	3.8	3¼	—	3¼	3.3	3
30	—	5½	3.2	3	6.0	3	3.2	3¼
45	—	4½	3 0	3	5.5	3	3.0	3
60	—	5½	2.7	2	4.9	2	2.5	2
75	—	5¼	2.7	2	4.1	2	2.6	1¾
90	—	4½	2.4	1½	3.3	1½	2.4	1¼

and 6.0 percent, respectively, the air contents at 60 minutes mixing time diminished only as far as 2.0 and 1.5 percent.

In the nominal 6-sack series the results are similar to those secured in the 4½-sack series, with some minor variations. With no addition and Admixture C, maximum air was entrained at the 2-minute mixing time. Thereafter entrained air decreased with increasing mixing time. In this richer mix, however, with both Admixtures A and B air contents reached a maximum only after 10 minutes mixing and thereafter decreased regularly. Here final air contents were considerably greater since mixing time was shorter after the maxima had been reached.

With or without the addition of an air-entraining agent, a maximum air content was attained in the early stages of mixing and thereafter decreased at a fairly regular rate. This decrease was fairly rapid until the minimum air content was approached and thereafter continued slowly or ceased. The actual air content at any given mixing time depends on the maximum air content attained. In the leaner mixes the maximum was attained rapidly, but with the richer fatter mixes maximum air contents, where they are high, were not attained until after a moderate degree of mixing in addition to the minimum mixing time. In the richer mixes entrained air was lost at a slower rate.

Data from the field series shown in Table 4 indicate that the laboratory mixer is probably much more effective in entraining air and subsequently decreasing air content with mixing time than would be most field mixers, particularly transit-mix equipment. The field mixes were made in transit-mix trucks operated at mixing speed for the first 15 minutes and then at agitator speed for the remainder of the time. All mixes had nominal cement factors of 6 sacks per cu yd and no water was added during mixing and agitation. In this series the maximum air content, in each case, was attained by the time the first determination was made at 15 minutes. Thereafter, a steady decrease in air content was observed but at a much slower rate than in the laboratory.

TABLE 5—AIR ENTRAINMENT IN RELATION TO BATCH SIZE

Size of batch cu ft	Air content, percent by volume			
	No addition	Admixture		
		A	B	C
Nominal 6 sacks per cu yd				
½	0.7	3.0	3.0	1.9
1	0.8	4.4	3.9	2.2
1½	1.0	4.5	4.1	2.4
2	1.0	4.3	4.1	2.9
2½	1.1	4.6	4.5	3.0
Nominal 4½ sacks per cu yd				
½	1.5	5.1	5.1	2.7
1	1.6	6.1	5.1	4.2
1½	1.5	6.1	6.0	3.8
2	1.5	5.8	6.0	3.5
2½	1.5	6.0	6.2	3.9

Varying size of batch

The effects of batch size for the type and size of mixer used are given in Table 5. In the nominal 4½-sack series the air contents with no addition show no significant variation with size of batch. In the nominal 6-sack series, with no addition, the air contents appear to be slightly lower with the two smallest batches. With Admixtures A, B, and C in the nominal 4½-sack series the smallest batch (½ cu ft) gave definitely lower air contents than the other batch sizes. With Admixture B this appears also to be true for the next smallest batch (1 cu ft). Similarly, in the nominal 6-sack series, all three air-entraining agents gave lower air contents with the smallest batch (½ cu ft). With Admixture C the next two smallest sizes (1 and 1½ cu ft) also appear to entrain somewhat less air. Otherwise air content varied little with batch size.

If the batch size is below a certain minimum, mixing is not effective and less air is entrained than in normal mixing. Visual observation of the mixing action was that there was little or no tumbling action with small batches. It may also be inferred, although no determinations were made with batches exceeding the capacity of the mixer, that with overloading mixing would not be vigorous and little air would be entrained. Batch size seems to have a greater effect on air entrained in the richer mixes, probably because they are stickier and require more vigorous mixing. This effect is most notable with Admixture C, again probably because Admixture C tends to make the mix more sticky.

Different cements

The types, air entraining characteristics in the A.S.T.M. mortar test, and surface areas of the cements used in the investigation of effect

of brand of cement on air entrainment are given in Table 2. Air contents, water-cement ratios, and compressive strengths at 28 days are given in Table 6. Water-cement ratio and strength data are included because the wide variations in these properties with different cements are of special interest.

With no addition the variations in air content are comparatively small. In the nominal 6-sack series there seems to be some indication that those cements giving higher air contents in the A.S.T.M. mortar test also entrain the highest percentages of air in concrete. To a smaller degree this is true also in the 4½-sack mix. For those cements giving not over 7 percent of air in the A.S.T.M. mortar test, there is no apparent correlation between the mortar test and the air content of the concrete. In the nominal 6-sack series, one cement (A) gave an exceptionally high air content and since this did not occur in the 4½-sack series, it seems probable that this was accidental or an error.

It is generally true that Admixtures A and B give higher air contents with those cements which give higher air contents in the A.S.T.M. mortar test. There are many exceptions to this general trend in both the nominal 6 and 4½-sack mixes. With Admixture C higher air contents were secured, as would be expected, than with no addition but lower than with Admixtures A and B. This is not significant since the proportions of the latter admixtures could be varied to increase or decrease the air entrained.

TABLE 6—INFLUENCE OF BRAND OF CEMENT ON AIR ENTRAINMENT, WATER-CEMENT RATIO AND COMPRESSIVE STRENGTH

Brand of cement	Air entrained, percent				Water-cement ratio, gal. per sack				Compressive strength psi			
	No addition	Admixture			No addition	Admixture			No addition	Admixture		
		A	B	C		A	B	C		A	B	C
Nominal 6-sack mix												
A	3.1	5.2	5.3	3.6	5.20	4.96	4.99	4.72	4950	4790	4350	5130
B	1.2	5.3	4.4	3.2	5.46	4.96	4.94	4.60	4830	4400	4690	5550
C	1.2	5.4	5.5	3.0	5.46	5.01	5.09	4.44	5410	4310	3910	6540
D	1.1	5.1	5.4	2.0	5.40	4.99	5.14	4.54	5140	4310	3850	6140
E	1.4	3.9	3.5	2.6	5.52	5.20	5.26	4.83	5070	5640	4940	6980
F	2.3	6.2	5.4	4.5	5.55	5.06	5.31	4.76	5430	4460	4100	6330
G	2.4	7.9	6.3	4.3	5.62	4.95	5.24	4.96	5020	3430	3560	5290
H	1.0	5.4	4.5	3.6	5.65	5.45	5.40	5.14	4480	3520	4260	5000
I	1.2	6.2	5.2	3.0	5.66	5.21	5.40	5.01	5090	4400	4350	6300
J	1.8	7.8	8.1	4.6	5.58	5.26	5.30	4.83	4940	4000	3610	5730
K	1.0	6.9	5.9	3.6	5.71	5.23	5.24	4.72	5360	4370	4770	6170
L	1.7	7.2	5.8	4.3	5.78	5.31	5.44	4.95	4610	3300	3940	4740
Nominal 4½-sack mix												
A	2.0	6.0	5.5	4.3	7.77	6.50	6.56	6.45	3440	2950	3980	3710
B	1.7	5.0	5.1	4.4	8.04	6.48	6.69	6.79	2850	2940	3230	3060
C	1.7	6.2	6.3	3.4	8.10	6.29	6.63	6.61	3410	3370	3670	3910
D	1.8	5.9	5.6	3.2	7.74	6.33	6.70	6.48	3320	3130	3200	4350
E	2.0	4.2	4.8	3.8	7.68	6.55	6.71	6.46	4330	4300	4610	5110
F	3.8	6.4	6.3	4.9	7.51	6.19	6.62	6.05	4240	3670	4120	5270
G	2.5	6.4	6.8	5.0	7.69	6.72	6.66	6.86	3580	3160	3080	3430
H	1.3	4.9	5.7	3.3	7.81	6.88	6.91	6.69	2760	2990	2950	3480
I	1.6	5.5	5.4	3.2	7.85	6.93	6.96	6.84	3340	3560	3360	3940
J	4.5	7.1	7.9	5.3	7.46	6.66	6.61	6.26	3130	3500	3180	4100
K	1.5	6.4	6.3	3.5	7.88	6.75	6.78	6.63	3390	3780	2990	4460
L	1.6	6.5	6.3	4.0	8.10	6.91	6.81	6.65	2630	2950	2570	3230

In general, it seems reasonable to expect higher air contents with no addition, and with Admixtures A, B and C in concrete mixes for those cements which give high air contents in the A.S.T.M. mortar test. In any particular case however, this may not be so. The A.S.T.M. mortar test, which was not intended for use with non-air-entraining cements, therefore cannot be taken as a criterion of the air contents to be expected in concrete mixes made with non-air-entraining cements, with or without air-entraining admixtures.

In general, cements which give high air contents in concrete mixes with Admixture A will also give high air contents with Admixtures B and C and in some cases with no addition. Because of a number of exceptions, however, neither the air contents secured with no addition nor those secured with Admixture A reflect the amount of air which will be entrained with Admixtures B or C, or other air-entraining agent. There appears to be no means of predicting the air entrained by an air-entraining agent with any particular cement prior to making up a mix with that agent.

One fairly consistent observation is that the air contents in the 4½-sack mix series are higher than those in the 6-sack series. This is in accord with previous observations* on the relation between cement factor and air entrainment.

The most noteworthy observation is the wide variation in entrained air between cements with the same proportions of air-entraining agent. There is considerable spread between the highest and the lowest air contents secured. This spread appears to be very nearly the same for no air-entraining agent and for Admixture C in the nominal 6-sack series while Admixtures A and B give much more variable results with the different cements. In the nominal 4½-sack series no addition, Admixtures A and B show approximately the same degree of variability, while Admixture C gives a smaller spread.

There is no recognizable relation between surface area and entrained air. For example, Cement G, which has the lowest surface area, in many cases gave air contents among the highest, whereas Cement B, also a low surface area cement, generally gave the lowest or nearly lowest air contents. On the other hand, Cement E, with the highest surface area excepting Cement F, frequently, but not always, had the lowest air content of all the cements. Cement L, a higher surface area cement, except for the Type III cements, frequently showed high air contents.

It will be apparent that if a concrete mix is made with a given cement to produce an air content of 4 percent then a change in brand of cement might produce an air content varying from 2 to 6 percent. This would

*Scripture, E. W., Jr. and Litwinowicz, F. J., "Some Factors Affecting Air Entrainment," ACI JOURNAL, Feb. 1949, *Proc.* V. 45, p. 433.

be true with any of the air-entraining agents tested, and in the richer mixes the differences with Admixtures A and B might be even greater.

There are considerable differences in water-cement ratio between the different cements. For example: the average difference between the highest and the lowest water-cement ratios, for no addition and for all three air-entraining agents in the 6-sack series is approximately .57 gal. per sack of cement and for the 4½-sack series is .65 gal. per sack of cement.

There appears to be little relation between the differences in water-cement ratio and type of cement or surface area. While in the 6-sack series the Type III cements with high surface areas required the most water, in the 4½-sack series they generally had slightly lower water requirements than the other cements. Again, Cement D, a Type II cement with higher surface area than the corresponding Type I cement of the same brand (Cement C), required about the same or slightly less water. Cement B, with the lowest surface area, required as much or more water than a number of cements with higher surface areas.

The compressive strength of concrete made with different brands of cement varied greatly. In the mixes with no addition there are variations of about 1000 psi in the 6-sack series and 1700 psi in the 4½-sack series. With Admixtures A, B, and C even larger variations of about 2000 psi were found. On the average the spread in strength was 1650 psi or about 35 percent variation from the average in the nominal 6-sack series and 1850 psi or 50 percent for the nominal 4½-sack series. The implication is that a change in brand of cement without re-design of the mix may seriously affect the strength of the concrete. This would be true whether or not an air-entraining agent was added, but the use of an air-entraining agent seems to affect this property more acutely.

CONCLUSIONS

1. With or without an air-entraining agent, a maximum air content is attained in the early stages of mixing and thereafter decreases with further mixing at a fairly regular rate. This decrease is fairly rapid until the minimum air content is approached, and thereafter continues slowly or ceases. The actual air content at any given mixing time will depend on the maximum air content attained. In the leaner mixes maximum air content is attained rapidly but with the richer fatter mixes maximum air contents, when high, are not attained until after a moderate degree of mixing in addition to the minimum mixing time.

2. As long as mixing is vigorous the air content of the mix is not significantly affected by batch size. With batches either too small or too large for the mixer, mixing will not be vigorous and air contents

will be low. This effect is slightly greater in the stickier, rich mixes than in the lean mixes.

3. There is a high degree of variability in entrained air, water-cement ratio and compressive strength with different brands of cement. Under ordinary circumstances, therefore, it is impossible to predict without making tests the air content, water-cement ratio or strength secured with any particular brand of cement. Further, there appears to be no correlation with surface area, type of cement or air entrained in the A.S.T.M. mortar test.

ACKNOWLEDGMENT

The field tests were conducted at the Superior Sand and Gravel Co., Phoenix, Ariz., and the cooperation of the staff of that plant is acknowledged with thanks. Thanks are also due to Dean E. Bryant, of The Master Builders Research Laboratories, for the A.S.T.M. mortar test air determinations and the surface area determinations on the twelve cements used in this work.

Promising results are indicated in this preliminary report on a method of determining the air content of hardened concrete.

A Method for Determining the Air Content of Fresh and Hardened Concrete*

By ROBERT P. VELLINES and THOMAS ASON†

SYNOPSIS

A method and equipment are described for determining the air content of fresh and hardened concrete based on the principle that air entrained in concrete reacts to external pressures and that volume changes follow the gas laws. The air entrained in hardened concrete reacts to an external pressure in the same manner as an equal volume of air would if it were not surrounded by hardened cement paste. The volume of the air in the specimen is proportional to the time required for a quantity of gas to escape through an orifice from a pressure of p_1 to a pressure of p_2 at a constant temperature.

INTRODUCTION

Two methods[1,2] for determining the air content of fresh concrete are based on the principle that air entrained in the plastic concrete reacts to pressure applied to the surface of the concrete and changes in volume in accordance with the gas laws. Heretofore, a method based on this principle has not been suggested for determining the air content of hardened concrete, probably because it was assumed that the air entrained in the hardened concrete could not react to a pressure applied to the surface of a specimen. Laboratory results indicate that the air in hardened concrete does react to an external pressure and that the air content of such concrete can be determined by such a method. Since many concrete technologists are interested in methods for determining the air content of hardened concrete and since it may require considerable work to perfect the equipment and technique required for a method based on this principle, it seemed desirable to publish the results of the preliminary experiments to permit those interested in such a method to contribute to its development.

*Received by the Institute Feb. 17, 1949. A brief presentation was made at the Research session at the ACI 45th annual convention, New York, N. Y., Feb. 25, 1949. Title No 45-40 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 9, May 1949, *Proceedings* V. 45. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Sept. 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich

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Boehler and Ason[3], in connection with the development of equipment for determining the surface area of cement, developed equipment for determining the density of solids using air instead of a liquid as the displacement fluid.

The equipment for the air displacement method consists essentially of an air source, valve and an air chamber fitted with an orifice and connected to a mercury manometer in which are suspended two electrodes of different lengths (Fig. 1). The air chamber and sample cup shown are not essential when using this apparatus for density determinations.

The principle of the method is that the volume of the system is proportional to the time required for a quantity of gas to escape through an orifice from a pressure of p_1 to a pressure of p_2 at constant temperature, or that

$$V_1 t_2 = V_2 t_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

In using the equipment, compressed air is admitted through the air valve to develop a pressure* in the system (air chamber, connecting tubing and manometer) such that the mercury rises in the manometer tube to a point above the end of the shorter electrode. The valve is then closed and, as air escapes through the orifice, the mercury drops in the tube. When the mercury falls below the end of the shorter electrode, it starts an electric timer and, as it drops below the end of the longer electrode, it stops the timer. Thus, the time required for the gas to escape as the pressure drops from that represented by the position of the end of the shorter electrode to that represented by the position of the end of the longer electrode is measured and recorded as t_1. The volume of the system is then decreased by inserting in the air tank a steel plug of known volume, V_3, and the operation is repeated to obtain t_2, the time required for the gas to escape over the same pressure range. Then, from Eq. (1)

$$V_1 t_2 = (V_1 - V_3)\, t_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

where

V_1 = original volume of system
$V_1 - V_3$ = V_2, volume of space remaining in the system after inserting steel plug
t_1 = time required for air to escape from system of volume V_1
t_2 = time required for air to escape from system of volume V_2

Since V_3, t_1 and t_2 are known, V_1 may be calculated from Eq. (2). Having determined V_1 by this means, the process may be reversed by placing a solid of unknown volume, V_s, in the air chamber and deter-

*The actual pressures used are dependent on the capacity of the orifice and the sensitivity of the timing equipment. In this apparatus, the orifice is a 0.0135-in. diameter axial hole through a brass pipe plug and the timing range is between 10 and 26 psi gage pressure.

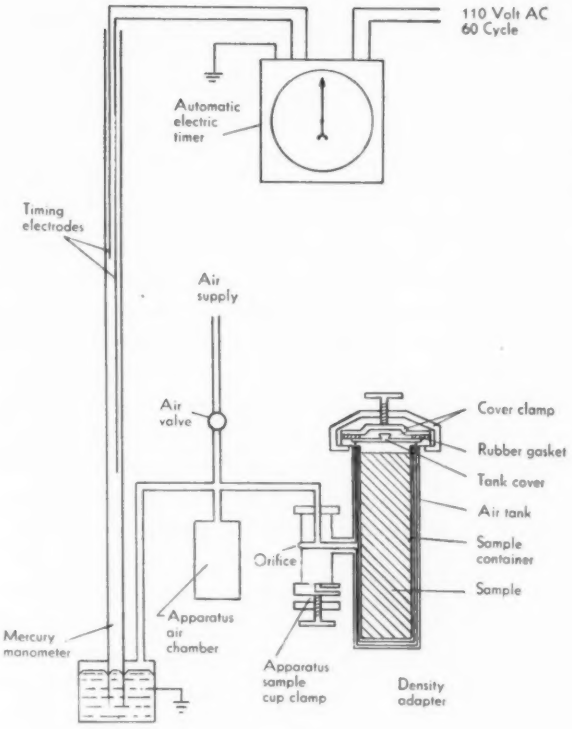

Fig. 1—Schematic diagram of density adapter and air permeability specific surface apparatus

mining t_s. Eq. (2) then becomes

$$V_1 t_s = (V_1 - V_s)\, t_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

EXPERIMENTAL PROCEDURE

This equipment and method were found to be very satisfactory for determining the volume of solids. Accordingly, it was decided that, if the air entrained in a hardened specimen of concrete would react to an external pressure, it should be possible to use this equipment to determine the volume of the unreacting portion* of a concrete specimen and, from this information plus supplemental data, to calculate the air content of the specimen. Since it is known that air entrained in fresh concrete reacts to an external pressure, it is obvious that the volume of the solids and liquid in a sample of fresh concrete may be determined by the procedure outlined above and that the difference between this volume and the original volume of the sample is the volume of the air in the sample. This reasoning appears to be borne out by the preliminary laboratory tests.

*In this case, the unreacting portion would comprise all material in the specimen except the air, *i.e.*, the solids plus water, whether the water is present as free water or in some other form, such as adsorbed water.

The testing program was devised to provide a comparison of the above method with A.S.T.M. Methods C185-47T and C138-44 and the Klein-Walker pressure method in tests to evaluate this method as a means of measuring the air content of hardened mortar and concrete specimens.

For the tests on mortar, a batch of mortar was prepared with a mix of 1:2.75 graded Ottawa sand by weight as described in the A.S.T.M. Method C109-47. One portion of this mortar was used for an air content determination by the C185-47T Method; another portion was used in the determination of the air content in the density apparatus; and two 2 x 2 x 9¾-in. prisms were molded from the remainder. The specimens were cured in the molds 24 hours in the moist cabinet. They were then removed from the molds, weighed and allowed to dry in the air of the laboratory for 3 days. The volume, V_s, of the solids (solid plus liquid) in each specimen was then determined in the density apparatus. The specimens were then weighed, immersed in water for 24 hours and then weighed under water and in the saturated surface-dry condition in air.

A similar procedure was followed for tests on concrete in which the coarse aggregate was No. 4 to ¾-in. gravel. Each batch of concrete was large enough to provide individual material for air content tests by Method C138-44, the Klein-Walker Method and the density apparatus method and for the molding of two 2 x 2 x 9¾-in. prisms. The concrete prisms were cured and tested in the same manner as the mortar prisms. The mortar and concrete specimens were then put aside to dry and, after 8 days in laboratory air, the determinations of air content in the density apparatus were repeated. The data obtained are given in Tables 1, 2 and 3.

DISCUSSION OF RESULTS

The total volume, V_t, of a hardened mortar or concrete specimen was calculated from the weights of the saturated specimen in air and under water. It was then assumed that this volume represented the summation of the volume, V_s, as determined by the density apparatus, the volume, V_{aw}, of the absorbed water and the volume, V_a, of the entrained air, thus

$$V_t = V_s + V_{aw} + V_a \quad \ldots\ldots (4)$$

For convenience, the volume in cubic centimeters of water absorbed is taken as numerically equal to the weight in grams of water absorbed and thus the volume, V_a, and the percent of entrained air are

$$V_a = V_t - V_s - V_{aw} \quad \ldots\ldots (5)$$

and

$$\frac{V_a}{V_t} \times 100 = \text{percent of entrained air in specimen} \quad \ldots\ldots (6)$$

TABLE 1—WEIGHTS, ABSORPTIONS AND VOLUMES OF FRESH AND HARDENED MORTAR AND CONCRETE

Specimen No.	Weight, grams: 24 hr.[1] W_1	Weight, grams: Dry[2] W_d	Weight, grams: SSD[3] W_s	Weight, grams: Immersed[4] W_w	Absorption,[5] grams V_{sw}	Volume, cc: Total[6] V_t	Volume, cc: Solid[7] V^*_s
4-day measurements—mortar							
A-1	1346	1265.5	1347.0	710.5	81.5	636.5	500.8
A-2	1346	1264.0	1348.0	711.5	84.0	636.5	497.5
B-1	1326	1257 5	1327 0	687.5	69.5	639.5	493.5
B-2	1320	1250.0	1321.5	684.0	71.5	637.5	489.8
C 1	1245	1188.5	1251.5	609.0	63.0	642.5	455.0
C-2	1235	1184.0	1242.0	603.5	58.0	638.5	454.3
4-day measurements—concrete							
D-1	1592	1538.5	1594.0	945.5	55.5	648.5	569.0
D-2	1605	1546.0	1605.0	949.0	59.0	656.0	573.3
E-1	1584	1529.5	1585.0	934.5	55.5	650.5	567.0
E-2	1581	1530 0	1581.0	932.5	51.0	648.5	567.5
F-1	1549	1503 5	1550.0	895.5	46 5	654.5	555.5
F-2	1536	1495.0	1537.0	887.5	42.0	649.5	560.0
13-day measurements—mortar							
A-1	1346	1264.0	1347.5	715.5	83.5	632.0	497.5
A-2	1346	1267.0	1348.0	716.5	81.0	631.5	499.5
B-1	1326	1262.0	1328.5	693.5	66.5	635.0	498.0
B-2	1320	1258.0	1322.5	689.0	64.5	633.5	494.5
C-1	1245	1199.0	1251.5	613.5	52.5	638.0	464.0
C-2	1235	1191.0	1243.0	608.0	52.0	635.0	460.5
13-day measurements—concrete							
D-1	1592	1545.0	1592.5	947.0	47.5	645.5	581.5
D-2	1605	1552.0	1605.5	951.0	53.5	654.5	581.0
E-1	1584	1534.0	1585.5	933.5	51.5	652.0	572.5
E-2	1581	1532.0	1582.0	934.5	50.0	647.5	570.0
F-1	1549	1502.0	1550.0	897.5	48.0	652.5	550.0
F-2	1536	1492.5	1536.5	888.5	44.0	648.0	555.0

[1] W_1 —initial weight when removed from mold.
[2] W_d —weight after drying either 3 or 8 days.
[3] W_s —weight in saturated surface-dry condition.
[4] W_w —weight under water after 1 day immersion.
[5] V_{sw} —weight assumed to be equal to volume in cc.
[6] V_t —volume calculated from W_s and W_w.
[7] V_s —volume of solids measured in density apparatus.
*Average of several determinations.

TABLE 2—RESULTS OF TESTS FOR AIR CONTENT OF FRESH AND HARDENED MORTAR

Batch No.	Air content of fresh mortar, percent: A.S.T.M method C 185-47T	Air content of fresh mortar, percent: Density apparatus	Specimen No.	Air content of hardened mortar, percent: Density apparatus: 4 days	Air content of hardened mortar, percent: Density apparatus: 13 days
A	8.3	8.7	A-1	8.5	8.1
			A-2	8.1	8.1
B	11.5	11.7	B-1	12.0	11.1
			B-2	12.0	11.8
C	20.6	21.2	C-1	19.4	19.0
			C-2	19.8	19.3

TABLE 3—RESULTS OF TESTS FOR AIR CONTENT OF FRESH AND HARDENED CONCRETE

Batch No.	Air content of fresh concrete, percent			Specimen No.	Air content of hardened concrete, percent	
	Klein-Walker	Gravimetric A.S.T.M., C138-44	Density apparatus		Density apparatus	
					4 days	13 days
D	2.1	0.9	2.2	D-1	3.7	2.6
				D-2	3.6	3.1
E	3.0	3.6	3.3	E-1	4.3	4.3
				E-2	4.6	4.2
F	7.7	7.5	8.4	F-1	8.0	8.4
				F-2	7.3	7.6

It may be seen from Tables 2 and 3 that the values obtained by this method for the air contents of the hardened specimens at the ages of 4 and 13 days agree fairly closely with those obtained on the fresh concrete. Thus, it appears that the air entrained in a specimen of the size used in this study reacts to an external pressure in the same manner as an equal volume of air would if it were not surrounded by hardened cement paste.

Attempts were made to determine the volume of the solids in the specimens at the age of 24 hours and in the saturated surface-dry condition but the results were erratic, probably because of the evaporation of water during the density test.

The variations between the values obtained for the air contents of the hardened specimen and of the fresh concrete are of about the same magnitude as are the variations in the air contents of the fresh concrete determined by different procedures. It is generally recognized that several samples of concrete taken from the same batch are likely to vary slightly. It is also recognized that the work done in placing the sample of concrete in the containers and molds may either increase or decrease the air content slightly. Hence, there is no way of being certain that a specimen molded from a given batch of concrete will have precisely the same air content as that of a sample of the concrete placed in the container of one of the instruments used for measuring the air content of the fresh concrete. Accordingly, comparisons such as those made in Tables 2 and 3 can only be used to show whether or not the air contents determined on the hardened specimens are within the range expected from the air contents determined on samples of the fresh mortar and concrete.

Sufficient tests have not been made with this method to permit a critical discussion of the data. Hence, no attempt will be made at this time to explain the rather large differences obtained at the ages of 4

and 13 days for the air contents of the B-1 and D-1 specimens. Also, the authors are not prepared to say whether or not the assumptions made in making the calculations are entirely justified. For example, it is known that liquids are compressed to some extent when adsorbed on solids. Powers and Brownyard[4] give values of about 0.88 for the specific volume of the total water adsorbed on hydrated portland cement pastes. No such decrease in the specific volume would be expected for the water absorbed by specimens which have been dried only in atmospheres of normal temperatures and relative humidities. Hence, the assumption that the volume, in cubic centimeters, of the water absorbed by the specimens in these tests is equal to the weight in grams, probably does not produce a significant error in the air content calculated for the dry specimens. Furthermore, during the absorption of this water, the specimens expand a little so that all of the space occupied by the absorbed water was not present in the dry specimens. If the air content of hardened concrete is to be referred to the air content of the fresh concrete, this expansion on absorption provides a volume, V_t, approaching the volume of the fresh material.

The data in this paper are of a preliminary nature, with the investigative work fitted to the dimensions of the existing test equipment but similar equipment on a larger scale could be constructed to accommodate larger specimens. It seems entirely possible to construct portable apparatus, based on this principle, suitable for use in the field and capable of measuring the air content of both fresh and hardened concrete. For simplicity, a dial gauge and stop watch might be used to replace the manometer and automatic timer.

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Economies designed into a structure may be lost when shortages force substitution of materials the use of which cause extra work. A study of objectionable characteristics of a particular material may indicate precautionary measures or a change in technique which will permit its satisfactory use.

Study of Dusty Concrete Ceilings*

By JACOB FELD†

SYNOPSIS

In a large eastern housing project of two-story concrete buildings considerable trouble was experienced from the "dusting" of first floor ceilings which were poured against new pressed board forms. The experimental work to isolate the cause, the results of the test and their interpretation indicated that the dusting was caused by an acidic resinous substance in the surface of the form board. The substance on the surface of the board appeared to be neutralized when the form was used for the first time, inasmuch as reused form boards gave little trouble. In concluding, the author recommends that new types of form material be checked for possible chemical reaction before use in large amounts.

INTRODUCTION

One of the important economies of reinforced concrete housing is smooth concrete ceilings which can be painted without any special preparation. When the concrete ceiling is not smooth and hard, expensive corrective work is necessary, whether in the form of laborious preparation of the concrete and plastering, or resurfacing by removal of the unsatisfactory base for painting. These extra costs void any economy of exposed concrete ceilings, especially after the extra money has been spent on smooth forms.

The development of smooth ceiling form technique began about 25 years ago. To eliminate costly hand grinding of normal form fins, many lining materials were tried. Among the unsuccessful materials, in the early 1920's, were newspapers, roofing paper, linoleum and metal sheets. Plywood soon became the standard material. However, when other uses made plywood unavailable for forms, hard-pressed composition boards were most commonly substituted. In a large two-story housing project, where the first floor slab was poured on rough lumber forms,

*Received by the Institute Dec. 20, 1948. Title No. 45-41 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 9, May 1949, *Proceedings* V. 45. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Sept. 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

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considerable trouble resulted from the use of a pressed board form for the second floors. This paper describes the experimental work done to determine the cause and to correct the difficulty.

GENERAL DATA

The project consisted of about 250 two-story narrow wall bearing concrete slab buildings, with a row of center columns and beams, the ceilings of the first story only designed as exposed concrete surfaces. Concrete aggregates were washed and properly graded river sand and gravel. Three different brands of cement were used, denoted herein as *P*, *H*, and *M*. The concrete batch, on the basis of dry aggregates, consisted of 470 lb cement, 1420 lb sand and 1810 lb gravel. Water was added to provide a 5-in. slump, using 5-cu yd transit-mix trucks for mixing and delivery. The 7-day strength of the concrete was 2000-2520 psi and slump varied from 5 to 6 in. Typical form construction and resulting ceiling surface immediately after stripping are shown in Fig. 1 and 2.

DIFFICULTY ENCOUNTERED

When this project was first inspected 19 floor slabs had been poured, but not all had been stripped, so that all the surfaces were not immediately visible.

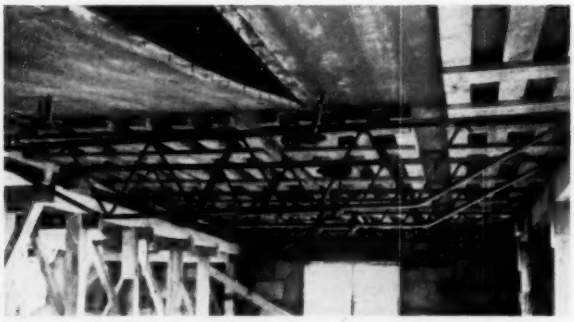

Fig. 1—Typical form supports

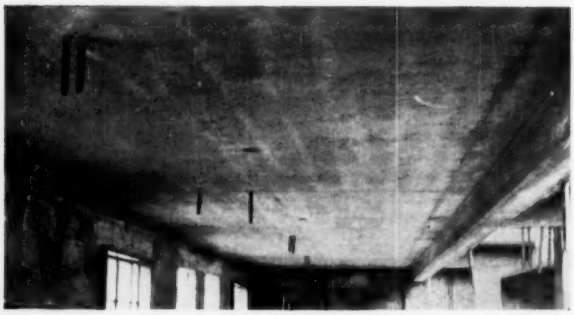

Fig. 2 — Typical ceiling after stripping of forms

In three buildings, using cement *M* and new form material, the forms were stripped after 2, 3 and 7 days, respectively. Concrete surfaces were completely covered with dust making the application of paint impossible. Brushing exposed the aggregate and spraying with water formed a gummy layer which could not be completely removed with spatula or wire brush.

Three other buildings in which the form material was new showed the same dusty surfaces although in two the forms were oil treated and in the other a pozzolanic material was added to the concrete.

In two buildings, with second use of forms coated with oil, little or no dust was formed.

TEST PANELS

To localize as far as possible the conditions which caused the objectionable concrete surface, 16 test panels were made up and filled with the same concrete from a single truck load. The panels were about 30 inches square with one side formed as a beam (Fig. 3). The chronological order of the tests and the results obtained are shown in Table 1.

All new treated or untreated pressed board panels produced considerable dust on the concrete surfaces whether the board was used as received, coated with various oils or treated with a cement wash. The only exception was a shellacked panel from which the concrete, although rough, showed no dust. Concrete poured on oil-coated panels of re-used treated pressed board, plywood, metal or wood was free of surface dust.

From these panel tests it was concluded that in every case where new treated pressed board was used as a form, no matter what the surface treatment, the resulting concrete was unsatisfactory. A chemical reaction between the wet concrete and some ingredient of the pressed board was indicated. Apparently, one use of the board removed or neutralized the agent which reacted with the wet concrete. Any other

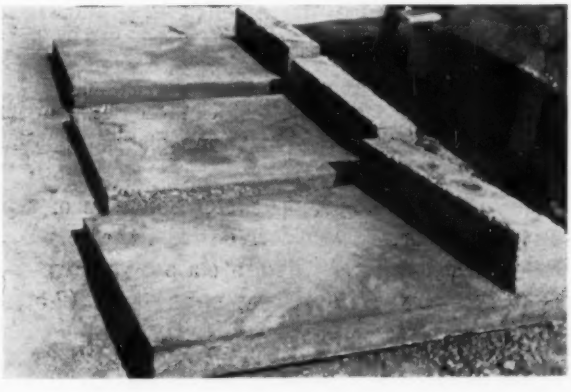

Fig. 3—Test panels showing dusty surfaces

TABLE 1—TEST PANEL FORM MATERIALS AND RESULTING CONCRETE SURFACES

Panel No.	Form material	Surface treatment	Condition of concrete surface
1	⅛-in. untreated pressed board	Shellac	Rough but no dust
2	¾-in. wood	S.O. 145 oil	No dust
3	¾-in. wood	King 85 oil	No dust
4	New treated pressed board	S.O. 145 oil	Some dust
5	New treated pressed board	No oil	Considerable dust
6	New treated pressed board	King 85 oil	Considerable dust
7	Old treated pressed board	S.O. 145 oil	No dust
8	New treated pressed board weathered for 4 days	S.O. 145 oil	Considerable dust
9	New treated pressed board	Shell 80 oil	Considerable dust
10	New treated pressed board	Carnia 245 oil	Considerable dust
11	New treated pressed board (concreted immediately after oiling the form)	S.O. 145 oil	Considerable dust
12	New treated pressed board	Cement wash on S.O. 145 oil	Considerable dust
13 a	½ new pressed board	S.O. 145 oil	Some dust
b	½ plywood	S.O. 145 oil	No dust
14 a	⅓ new pressed board	Carnia 245 oil	Considerable dust
b	⅓ new pressed board	Cement wash	Considerable dust
c	⅓ metal covered	½ oiled ½ cement wash	No dust No dust
15	New treated pressed board	No oil	Considerable dust
16 a	½ new treated pressed board	King 85 oil	Some dust
b	½ plywood	King 85 oil	No dust

type of form, with the same concrete mix and similar oil treatments, gave satisfactory surfaces, as far as dusting was concerned. The only exceptions were found on some of the beam sides near the slabs, where plain wood forms had been used, but undoubtedly the cause was spillage or leakage of liquid from the slab form down the beam sides.

Since the job was in full progress and substitute form material was not available, an immediate cure was necessary. It was deduced that the active agent could be neutralized by cement, inhibited by covering with shellac and possibly also neutralized by treatment with lime wash.

CHEMICAL TESTS

Representative samples of the dust were collected from the forms and from the concrete ceilings and together with some of the cement were chemically analyzed (Table 2).

The calcium, iron and alumina, and siliceous matter contents of the powders indicated that the greater portion was derived from the cement. However, the powders did not harden when formed into a pat with water.

Another extract of the powder showed variable amounts of oil (2.5 to 5 percent). After the oil was extracted, the powder was treated with hydrochloric acid and water and the resultant solution was extracted

TABLE 2—ANALYSES OF DUST AND CEMENT

Percent	Powder from form	Powder from ceiling	Cement
Loss on ignition	8.23	8.12	0.65 gain
Acid insoluble matter	21.41	52.15	21.31
Calcium (CaO)	48.77	26.67	63.93
Iron and alumina	18.03	10.90	10.26
Magnesium (MgO)	3.03	2.12	2.31
Sulphates (SO_3)	0.53	Trace	1.53
Chlorides	Trace	Trace	None
Total acid soluble	70.36	39.69	78.03

with ether. The final residue was a small amount of reddish brown resinous matter. An ether extract of oiled samples of the form material also showed some resinous substances, as did also an extract of unoiled form material. The resinous material obtained from the extractions was distinctly acid.

The chemical explanation of the dust formation was that the resin acids of the pressed board were brought to the surface by the oil treatment. Contact with the alkaline calcium hydroxide of the cement in the concrete mix caused the formation of calcium salts of the resinous acids. These salts are highly insoluble and together with the oil as a dispersion medium may mechanically coat other particles of cement and increase the amount of inert powder. Once the resinous substance in the surface of the form board had been removed by one use of the form, further uses caused no more dust. The resinous materials also may have been present in the body of the board, but were probably inhibited by the chemical change on the surface.

The writer has noted similar reactions between cement and resins in some cement-stabilized soils where the soils contained considerable amounts of rotted vegetation.

FULL SCALE INVESTIGATION

To check the conclusion that the form material was the sole cause of the trouble and that only the first use of the form would result in dust formation, inspections were made of other housing projects where the same material was being used, and a program of full scale concreting with different form materials was initiated.

An inspection of one project, where buildings progressed in groups of five, showed only a few ceilings with dust in serious amounts. The forms had been re-used three and four times and oil was applied as close to the date of concreting as possible. Apparently dusting resulted only from the initial use of the form material.

Other multi-story projects reported no trouble. However, since the forms were first used in the basement where the ceilings were not painted, the lack of reported trouble was understandable.

In the full scale tests, the center part of each floor slab was formed with new panels, and each end section with re-used panels. When stripped, the center was very dusty and soon assumed a dark brown-green color. The end sections were slightly dusty immediately adjacent to the center section but otherwise were dustless and much lighter in color. Other buildings were likewise formed with both used and new materials in longitudinal strips and in all cases the dust appeared only where new material was used for forms.

CONCLUSION

The test results proved that satisfactory concrete surfaces could not be obtained when new pressed board forms were used. In this project, considerable corrective work was necessary on the ceilings poured on new forms, although some relief was obtained by treating the new material with lime wash. It is recommended that any new form materials be checked for possible chemical reaction with concrete before use in large amounts. Undoubtedly chemical sealing of the acid ingredient is possible and could have been developed had time permitted.

In September **Letters from Readers** replaced **Job Problems and Practice,** with broader scope, more flexible form, and we hope more adaptability to the thoughtful, busy reader's use, as a forum for opinions, experiences and inquiries within the field of ACI activity. A letter does not need the studied treatment of a major paper to present valuable ideas for ready access to kindred minds.

Letters from Readers*

BY WAY OF SYNOPSIS

M. HIRSCHTHAL cites the Joint Committee Report for data on the effects of lubricating oil on concrete.

The application of sheet-iron models to problems of indeterminacy is described by RAFAEL BONNELLY G.

A reader asks for a practical method of bleaching a concrete structure.

A more mathematically correct method of compensating for free moisture in aggregates is proposed by HENRY COMACK.

Effect of Oils on Concrete (LR 45-16)†

M. HIRSCHTHAL, concrete engineer, Hoboken, N. J. advises.

I note in the March ACI JOURNAL that the question raised by Mr. Wening is not answered. Attention is invited to Appendix 1 of the Joint Committee Report which gives the effects of various substances on portland cement concrete and the suggested treatment where necessary. Among these substances are various oils and similar liquids. That part of the appendix relating to oils is quoted:

Substance	Effect on Unprotected Concrete‡
Light fuel oils above 30° Baume	None—some loss from penetration
Volatile oils—kerosene, benzine, naphta, gasoline	None—considerable loss from penetration
Heavy oils—30° Baume or Heavier	None—Very slight penetration

This may give Mr. Wening the information he desires, although lubricating oils vary greatly in character, viscosity, etc.

*A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 9, May 1949. *Proceedings*, V. 45. Separate prints are available at 35 cents each. Address 7400 Second Boulevard, Detroit 2, Mich.

†See also ACI JOURNAL, Mar. 1949, p. 555.

‡Loss referred to is loss of oil, not loss of strength by concrete.

Sheet-Iron Models in Structural Design (LR 45-17)

RAFAEL BONNELLY G.* describes an inexpensive test procedure.

The use of galvanized sheet-iron models for solving indeterminacy in rigid structures has advantages over other known model principles because of its low cost and the ease with which the apparatus can be built. The galvanized sheet-iron model, because of its greater width, may be used to solve problems with variable distributed loads imposed by jelly strips of variable sections placed on top of the model.

The apparatus consists of a balance, weight pans, wire hooks made out of paper clips, and pieces of fine copper wire, as shown in Fig. 1.

The procedure is as follows:

1. Cut the developed structure from a piece of sheet iron making the width proportional to the moment of inertia, and bend the cut model to the shape of the structure or frame. Then place one end B on the toothed plate of the balance, and suspend the other end A by a wire V_A from the handle, turning the latter to bring end A into a relative position with B.
2. Hook end A, by wire H_A, to the lower end of the lever of the balance.
3. Place balance pans and hook another pan in the point where load is to be applied.
4. Keeping wire V_A practically plumb, and wire H_A horizontal, check distance between ends A and B, to determine structure's span.
5. Now, move indicator (two pieces of fine wires nailed to a piece of wood) so that the wires just touch end A of the model.
6. Place a small weight W_B on the balance pan, which will cause the withdrawal of end A from the indicator wires.
7. Now, place enough weight W on the load pan, so as to return the end A to its original position, slightly touching the wires of the indicator.
8. If we call P the load in the structure, and H_A its original pull, then

$$\frac{P}{H} = \frac{W_P}{W_B}$$ from which the value of H_A is determined.

9. Once H_A is found, the vertical reaction V_A at end A may be found, and by doing so the moment in any point of the structure due to the load P may be also found by simple statics.

This apparatus, with a slight modification at end A, gives the solution for moments, vertical reactions and horizontal pull for fixed end structures. Photos of this use of the apparatus are available from the author.

Bleaching of Concrete by Weather (LR 45-18)

A reader asks for information on bleaching concrete and change of color with time.

A large monolithic concrete building, whose exterior surface was originally rather dark in color, is now uniformly of a pleasing light color on the south and west faces. The north and east faces, however, for

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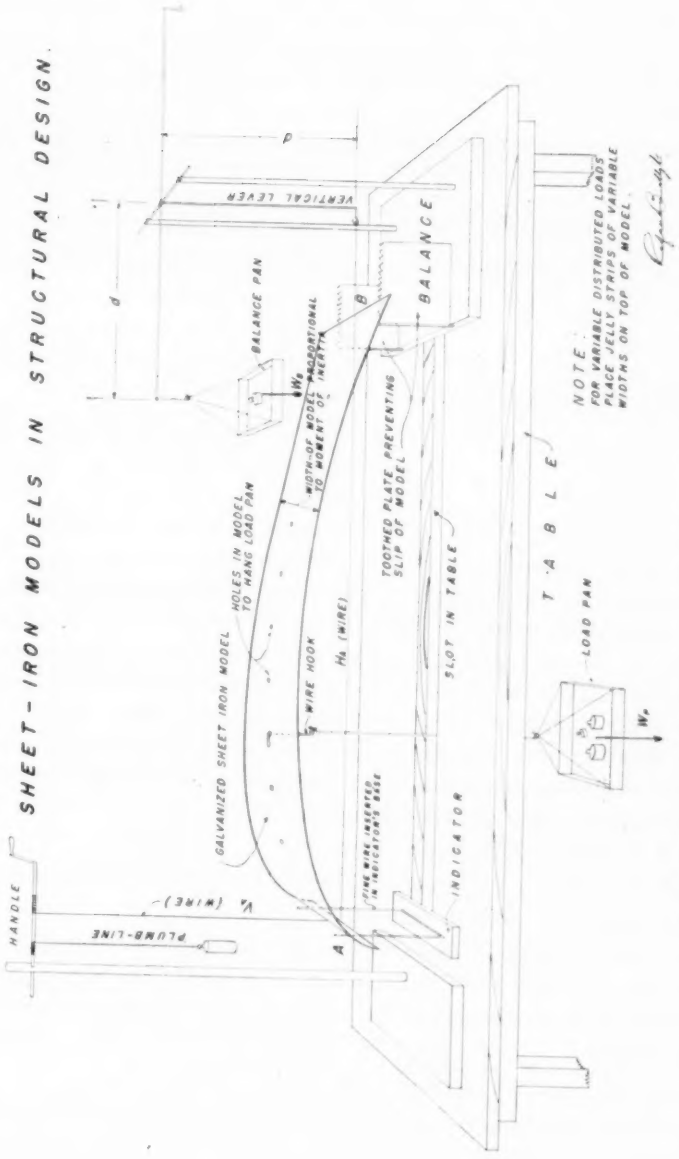


Fig. 1—Diagram of apparatus for study of sheet-iron structural models

many years have retained almost the original dark color except where streaked with water draining down from the top or from projecting ornamentation. (It is understood that the ornamental portions of the concrete contained a lime admixture.) The building is in a region of light rainfall, which comes principally from the southwest; and it is believed that the north and east faces are seldom thoroughly wetted.

It appears that the lightening in color has been caused by carbonation of lime which was originally at the surface or which was brought there by leaching or drainage. If so, would it be practicable to bleach the dark surfaces by periodically wetting them down with a hose? Approximately how much time would be required? Any information regarding the change in color of concrete with time would be appreciated.

Making Corrections for Free Moisture (LR 45-19)

HENRY COMACK, civil engineer, Flushing, N. Y., notes a discrepancy in conventional methods of allowing for moisture in aggregates.

The writer invites attention to the mathematical inaccuracy of the method generally used in correcting batch weights to compensate for the free moisture in the aggregates. This error becomes more significant in areas where high moisture contents are typical. To illustrate, let us correct by the conventional method a quantity of fine aggregate.

Assume 2000 lb per cu yd batch and average free moisture of 8 percent.

Then $2000 \times 0.08 = 160.0$ lb of water in aggregate.

$2000 + 160 = 2160$ lb corrected batch weight aggregate

But the additional aggregate also includes 8 percent free moisture, a factor not given adequate consideration, thus:

$160.0 \times 0.08 = 12.8$ lb of water

$12.8 \times 0.08 = 1.0$ lb A further correction is insignificant.

Total water 13.8 lb

Adding 13.8 lb to the former "corrected" weight of 2160.0 lb gives 2173.8 lb, the true weight of batch containing 2000 lb of dry aggregate.

A simple method of obtaining this result is suggested by the formula:

$$\frac{\text{Dry weight}}{(100.0 - \text{moisture})} = \frac{2000.0}{(100.00 - 0.08)} = 2174 \text{ lb}$$

No doubt this discrepancy is known but the experience of the writer indicates that the conventional method is widely used. A reason given is that such small weights would be insignificant. The writer contends, however, that it is the duty of concrete engineers and technicians to furnish mix designs for batching that are mathematically and analytically incontestable.

Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

Some new electricity generating stations

Concrete and Constructional Engineering, (London), V. 44, No. 2, Feb. 1949, pp. 65-69
Reviewed by GLENN MURPHY

Brief notes are given of the concrete work involved in the construction of eight power plants either completed or near completion in Britain.

The analysis of fresh concrete

R. H. H. KIRKHAM, *Concrete and Constructional Engineering*, V. 44, No. 2, Feb. 1949, p. 54
Reviewed by GLENN MURPHY

The method described is essentially that developed by Dunagan and reported by him in *Bulletin* 113 of the Iowa Engineering Experiment Station. The article contains pertinent comments on other mechanical and chemical methods of analysis.

Stability of SR-4 electric strain gages and methods for their waterproofing and protection in field service

A. BOODBERG, E. D. HOWE and B. YORK, *Transactions*, A.S.M.E., Nov. 1948, V. 70, pp. 915-22
APPLIED MECHANICS REVIEWS Jan. 1949 (Bernhard)

Experiments with SR-4 electric strain gages are described to study their stability with respect to changes in temperature and humidity. Most of the gages were attached to steel blocks during the tests and not subjected to loads. Several waterproofing agents were tested. The results indicate that under the above conditions the stability of the gages can be maintained over long periods of time.

Tests of prestressed concrete beams

Concrete and Constructional Engineering, V. 44, No. 2, Feb. 1949, pp. 52-53
Reviewed by GLENN MURPHY

Tests of two prestressed concrete beams by the Eastern Region of British Railways are reported. One of the beams was identical with those made for overline bridges of British Railways, containing eighteen 0.2-in. diameter steel wires in the bottom flange of an I-section 14 in. deep. The calculated load at failure was 8.3 tons, with failure in compression. After several preliminary loadings the beam failed at a load of 8¼ tons. A second beam with twelve additional unstretched wires in the tension flange was loaded to 9 tons several times. In each beam the loads were applied at two points near the center of a 26½-ft span.

*A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 9, May 1949 *Proceedings*, V. 45. Address 7400 Second Boulevard, Detroit 2, Mich.

The development of prestressed concrete

P. W. ABELES, *Civil Engineering*, (London), Jan.-Oct., 1948

APPLIED MECHANICS REVIEWS Jan. 1949 (Durelli)

A general review of prestressed concrete principles, properties, methods and calculations. Although no results of new tests are shown, the field is covered thoroughly and the bibliography is up to date. The author emphasizes the difference between pre-tension and post-tension prestressed concrete, and compares the different techniques in use from several points of view. An analysis is made of several applications, and general conclusions are drawn.

How to build a concrete road dead flat

CHARLES R. WATERS, *Concrete Highways and Public Improvements*, V. 30, No. 1, Spring 1949, pp. 10-13

This article describes New York State's method of securing smooth-riding surfaces on the newly constructed concrete Gardenville-Williamsville Highway.

The use of smoothing templets, scraping straightedges or lutes, were required to eliminate surface irregularities that often occur back of the mechanical longitudinal float. Both the wooden and steel lutes used in the operation were checked on a master straightedge twice a day. A burlap drag, drawn longitudinally over the pavement surface, gave the pavement the final finish. The lutes removed surface irregularities and also simplified the work of the finishers.

Lightweight concrete (Betons legers)

J. CHEFDEVILLE, *Cahiers du Batiment*, (Paris), Jan. 1949

In this article the author reviews the different types of lightweight concrete including those made of no-fines aggregate, pumice, slag, pozzolanic materials, cinders, clinkers, crushed slag, burned slate, expanded clay and sawdust. These types are first classified from a physico-chemical point of view; composition, apparent density and granulometry. A general comparison of lightweight concretes, with or without fines, makes it possible to establish the differences existing between their essential technological properties of compressive and tensile strength, thermal conductivity, capillarity, shrinkage and elasticity modulus. In conclusion the advantages of these types of concrete with regard to particular uses are enumerated.

The Sando and Klockestrand bridges

Concrete and Constructional Engineering, V. 44, No. 1, Jan. 1949, pp. 29-34

Reviewed by GLENN MURPHY

The Sandö bridge across the main channel of the Ångerman River in Sweden is the largest reinforced concrete arch span in the world, having a span of 866 ft. The other arm of the river is spanned by a continuous girder concrete bridge. Each bridge has a 39 ft 4 in. roadway and two 4 ft footpaths. The Sandö span is of cellular construction. Concreting was started using a timber arch for centering, but upon completion of the bottom slab of the concrete arch the timber arch collapsed. Failure was attributed to the reduction in strength of the timber because of its high moisture content. Erection of the concrete span continued after new timber centering with intermediate supports was erected. Some design details are included.

The stresses of shrinkage in compound structures of fresh and hardened concrete (Die Schwindspannungen in Verbundkorpern aus Neu- und Altbeton)

F. PFEFFER, *Oestrreichische Bauzeitschrift*, (Vienna), V. 4, No. 1, Jan. 1949, pp. 7-14

Reviewed by RUDOLPH FISCHL

Knowledge of the effect of shrinkage in fresh concrete poured between hardened

concrete members is important in providing continuity in precast constructions as well as for reconstruction work.

The author presents a well founded theoretical investigation concerning the shrinkage stresses in joints of different types and various sizes. An approximate method of reinforcing the fresh concrete in large joints is given and the application shown in different examples. The article emphasizes the importance of considering the shrinkage between fresh and hardened concrete in every compound structure and to take it into account together with the other working stresses. Eleven illustrations are included.

Prestressed concrete roof beams at Tiverton

Concrete and Constructional Engineering, V. 44, No. 2, Feb. 1949, pp. 50-52

Reviewed by GLENN MURPHY

Six prestressed concrete beams were used in the construction of an extension to weaving sheds at Tiverton, Devon, England. Each beam is 54 ft 7 in. long and is of I section, varying from 33 in. in depth at the center to 39 in. at the ends. The sections are 12 in. wide at the top and 7 in. at the bottom. The reinforcement in the bottom flange consists of forty 0.2 in. diameter cold-drawn steel wires embedded in a flexible metal sheath 4½ in. x 2 in. in cross section. Two ⅞-in. mild steel bars are used as reinforcement in the top flange. The concrete used in the beams had a cube strength of 7420 psi at 14 days. These beams are the first constructed in England using the Magnel-Blaton system of prestressing.

Reinforced concrete (Zelezovy beton)

F. KLOKNER, K. HRUBAN and A. PITRINEC, *Ceska matice technicka, Technicky pruvodce*, No. 24, 1947, Prague

Reviewed by IVAN M. VIEST

This reinforced concrete handbook is presented in three parts: (1) Materials and construction elements, (2) Design, and (3) Tables.

The first part includes a list of current Czechoslovakian specifications for plain and reinforced concrete, discussion of various elements of reinforced concrete structures, loadings, design of concrete mixtures and descriptions of the standard tests of materials.

The second part presents in detail the design of reinforced concrete structures based on the elastic theory. One chapter is devoted to ultimate design theories with detailed presentation of the current design practice in USSR as specified by the Russian specification OST 90003-1938.

The effects of electrolysis upon the strength of reinforced concrete

G. MOLE, *Engineering*, V. 166, No. 4319, London, Nov. 5, 1948

HIGHWAY RESEARCH ABSTRACTS

This investigation was undertaken by the British Electrical and Allied Industries Research Assn. to obtain confirmation of Bureau of Standards work and to extend the study to cover the effects of alternating-current electrolysis and the influence of both direct- and alternating-current electrolysis upon bond under conditions where there is no visible change at the surface of the reinforcing.

Conclusions are drawn concerning the effect of alternating-current and direct-current electrolysis on the electrical resistance of reinforced concrete and on the bond between steel and concrete. Methods used are described and the effect of the chemical composition of the concrete is discussed.

Handbook of Chemistry and Physics

The Chemical Rubber Co., 31st Edition, 2756 pp., $6.00

The 31st Edition of the Handbook of Chemistry and Physics adds much new material to this authoritative reference book. The title scarcely does justice to its scope. Of

particular interest to engineers are complete mathematical tables; properties and physical constants of the elements; tables of specific gravity; properties of matter; heat and hygrometry; sound; conversion tables; and a vast amount of other important data. A table of logarithms of decimal fractions has been added; also a table of squares of the sine and cosine and their product. The former table of arrangement of electrons in orbits has been replaced by a table of electronic configuration of the elements. In step with the trend toward more intensive study of nuclear physics, a table of isotopic masses has been included; also a table of wave lengths of the principle lines in the emission spectra of the elements. This table is taken from data only recently available. Complete revisions and new material added to the handbook amount to 180 pages.

Vacuum concrete

K. BILLIG, *Institute of Civil Engineering Journal*, 1948, No. 29, pp. 243-69
Building Science Abstracts V. XXI, (New Series), No. 10, Oct. 1948

HIGHWAY RESEARCH ABSTRACTS
(Mar. 1949)

In this report, based on research conducted mainly by the Russian Building Research Station, a historical and bibliographical review is followed by an account of the technology and properties of vacuum concrete and the equipment needed for vacuum treatment. Vacuum concrete technique is most suitable for the production of horizontal concrete slabs covering large areas (floors, roads), and the construction of concrete walls, partitions, thin facings of concrete, and thin suspended floors. The number of molds required for ordinary concrete can be greatly reduced by the rapid set and subsequent hardening of vacuum concrete. It is emphasized that the technology of vacuum treatment of concrete is still in the experimental stage.

Cement transportation in bulk (Transport du ciment en vrac)

J. SILLARD, *Cahiers du Batiment*, (Paris), Jan. 1949

For the last twenty years in France, cement has generally been delivered in paper bags. This unrecoverable packing is very practical for short distance deliveries. The shortage of paper and especially of Kraft paper has made it necessary to deliver in bulk, which has often proved unsatisfactory.

American methods of transportation in bulk should be adapted proportionally to French needs. It would seem that for an average enterprise, transportation within a radius of 50 to 100 km could be handled in 4 to 5-ton containers loaded on trucks used at the same time as silos or, even better, in trailer-containers.

When there are no cement plants within this radius, it might be advantageous, in badly war-damaged areas, to establish auxiliary allocation centers supplied in bulk from the nearest cement plants by shuttle trains or coasting boats.

Cellular concrete (Les betons cellulaires)

G. DEMARRE, *Cahiers du Batiment*, (Paris), Jan. 1949

Although cellular concrete has not been much favored in France until recently it is gradually becoming more popular in French building technique. The Centre Scientifique et Technique du Batiment has therefore established acceptance standards with respect to cellular concrete blocks used in the construction of walls, fillings and partitions.

There are two types of cellular concrete; sponge concrete (Schwemmbeton) and gas concrete both of which have been used on a large scale in Germany and Scandinavian countries. The manufacturing process for sponge concrete apparently produces more homogeneous products than that of gas concrete.

Investigation has established that in most countries there are at present very few regulations concerning the use of cellular concrete. Mention is made of the tests carried out at the Bureau Veritas with a view to establishing specifications. This work justifies the temporary provisions adopted for the control of cellular concrete.

Prestressed steel-tile structures

CHR. OSTENFELD and W. JONSON, *Ingenioren*, (Copenhagen, Denmark)
V. 58, No. 2, 1949, pp. 49-56 Reviewed by EIVIND HOGNESTAD

Prestressed steel-tile construction was invented in Switzerland. The basic members are prestressed beams consisting of a number of tiles with longitudinal grooves in which steel strings are encased by means of vibrated cement mortar. Shop-made beams of this type are used as reinforcement in masonry, over windows, etc., as well as for reinforcement in concrete floors. The effective prestress in the strings used at a Danish plant is 140,000 to 150,000 psi. This corresponds to a compression in the tiles of 200 to 1400 psi.

The basic assumptions of design are outlined as adjusted to the Danish specifications for reinforced concrete. Some tests are presented briefly. The production of beams is described in some detail followed by some information regarding a 330 x 110 ft roof structure.

One of the most important features of steel-tile construction is that reduced steel consumption and the absence of slab forms represent a considerable saving as compared to other slab structures.

The Tummel-Garry Hydro-Electric Works I

Concrete and Constructional Engineering, V. 44, No. 1, Jan. 1949,
pp. 9-17 Reviewed by GLENN MURPHY

This article in the series on hydroelectric development in Scotland contains a description of Clunie Dam, together with a surge chamber and a 1¾-m. tunnel. The project, which is estimated to cost about $24,000,000, includes three generating stations operating under heads of 606 ft, 173 ft and 48 ft and will supply 147,000 kw. Clunie Dam, across the Tummel River at Clunie, is a 37,000 cu yd concrete gravity structure with a maximum width of 90 ft and a length of 380 ft. The spillway is about 70 ft above the river bed, and a fish ladder is to be incorporated into the structure. Construction is proceding behind cofferdams which restrict the river to one side of the channel. The tunnel from the dam to the power house is approximately 23 ft in diameter and is concrete lined. The surge chamber, which has a diameter of 110 ft and is 100 ft deep, is excavated in rock and has an 18 in. minimum reinforced concrete lining. Design and construction details are included.

The Tummel-Garry Hydro-Electric Works-II

Concrete and Constructional Engineering, V. 44, No. 2, Feb. 1949,
pp. 61-64 Reviewed by GLENN MURPHY

This, the third of a series of articles on hydroelectric developments in Scotland, describes the construction of the concrete dam at Pitlochry. The gravity structure, which is 475 ft long, is to contain 43,000 cu yd of concrete, and auxiliary works will contain an additional 19,400 cu yd. The construction is similar in many respects to that of Clunie dam, described in a previous installment.

The acoustic strain gage

New Zealand Engineering, V. 3, No. 12, Dec. 10, 1948 HIGHWAY RESEARCH ABSTRACTS

The principle of operation of the acoustic gage (vibrating wire gage) is that the change in frequency of vibration of a stretched steel wire clamped to the structure under

test, gives a direct measure of the change in strain occurring in the structure. The stretched wire of the test gage is plucked electromagnetically and the note emitted is matched against the frequency of the note emitted by a second steel wire, the tension of which can be varied by a calibrated control. The matching is done by reducing the beat frequency of the two notes to zero.

Acoustic gages have been used to investigate the strength of highway bridges where they proved particularly valuable for measuring strains in bridge girders and masonry arches, as the observations could be made without interfering with the traffic passing beneath the bridge. Acoustic strain gages have been used in loading tests of reinforced concrete frameworks and steel roof trusses.

Dynamic strains have been recorded with acoustic strain gages by observing the change in beat frequency with an oscillograph and recording it with a movie camera. The photographic records obtained can be analyzed to give data for plotting strain-time curves.

Freeze and thaw durability of air-entraining concrete using Indiana aggregates

JACK B. BLACKBURN, Highway Research Board Paper,
28th annual meeting, December 1948 HIGHWAY RESEARCH ABSTRACTS

This paper contains a study of the freeze and thaw durability of seven Indiana aggregates used in concrete with varying air contents. The purpose was to determine the effect of air entrainment on the durability of these aggregates chosen for their absorptive and performance characteristics as being fairly representative of Indiana aggregates.

The conclusions presented are: 1. There is an optimum air content above which durability is not increased and flexural strength is considerably reduced. 2. As the degree of saturation of an aggregate increases before mixing, the durability of the resultant concrete decreases, all other variables being the same. 3. As the degree of saturation of a concrete mass increases the durability in freeze-and-thaw testing decreases. 4. The degree of saturation of a concrete mass decreases with an increase in air content. 5. The yield of air-entraining mixes is less than expected when the mix is designed as in this study. 6. The loss of yield results in an excessive cement factor.

Durability of portland cement concrete determined by primary directional freezing and uniform thawing

R. E. BOLLEN, Highway Research Paper,
28th annual meeting, December 1948 HIGHWAY RESEARCH ABSTRACTS

This paper covers a portion of an investigation of a method for determining the soundness of concrete prisms by primary directional freezing and uniform thawing. Most conventional freezing-and-thawing tests follow a procedure in which the change in temperature of the specimen proceeds fairly uniformly from all of the outside surfaces of the specimen toward the center of gravity of the smallest cross-sectional area. This procedure duplicates the direction of the change in temperature in some concrete such as bridge rails and exposed parapets, but does not duplicate the direction of the change in temperature of concrete pavement. Excluding joints, cracks and sides of the concrete pavement, the direction of freezing under normal exposure is from top to bottom of the pavement.

The data indicates that the directional freezing and uniform thawing is a rapid and severe test and that more uniform results can be obtained by using beams which have been cured for a minimum of 28 days, then by beams which have been cured only 7 days.

Model tests of Santa Luzia dam

MANUEL ROCHA and J. LAGINHA SARAFIM, Ministerio Das Obras Publicas, Laboratorio de Engenharia Civil, Lisbon, 1948 Reviewed by S. D. BURKS

The Santa Luzia Dam, completed in 1943, is the first important concrete arch dam built in Portugal, and therefore aroused considerable interest among designers and civil engineers in that country. This interest developed the desirability of comparing the results of deflection measurements in the structure with those determined analytically and measured in a test model. The dam is a thin arch approximately 70 m high, has a crest length of 115 m, and an upstream radius of 60 m. The thickness at the base is 12 m and is 2.5 m at the crest.

Experiments with concrete, celluloid, rubber, plastic and a mixture of plaster and diatomite demonstrated that the latter material would best satisfy the requirements for model material insofar as performance similar to the prototype was concerned. A scale of 1:100 was chosen for the model. Hydrostatic pressure on the model was obtained with mercury contained in a rubber bag confined in the 1 cm space between the upstream face of the model and the reinforced concrete form used for casting. Deflections were observed on the dam and foundations using Roch dial micrometers; strain measurements were determined by acoustic strain gauges.

The results of these tests indicated that (1) the behavior of the structure is greatly influenced by its asymmetry and by the deformation of the wing wall abutment, (2) the stress diagrams are in harmony with the asymmetry of the structure, (3) there was very satisfactory agreement between the radial deflections measured on the model and those observed on the prototype; there was substantial difference between the deflections calculated analytically by the independent horizontal arch method and those measured on the model and the prototype, (4) the directions of the principal stresses, at the points for which comparison was possible, were in good agreement, and (5) the magnitude of the horizontal stresses observed on the dam agree much more closely with the values measured on the model than with those computed analytically.

The prediction of temperatures in mass concrete by numerical computation

A. D. ROSS and J. W. BRAY, *Magazine of Concrete Research*, (London), No. 1, Jan. 1949 AUTHORS' SUMMARY

The effects in mass structures of the chemical generation of heat in the concrete are briefly reviewed. Some reference is made to the practical techniques designed to reduce steep temperature gradients and hence to control cracking due to thermal causes.

There remains, however, the problem of predicting temperatures in a mass of concrete of known thermal properties which is placed under specified conditions. Formal mathematics can give solutions only in the simplest cases but the number of influential factors in practice necessitates the employment of approximate step-by-step methods of numerical calculation.

The method proposed by Schmidt for the unidimensional flow of heat is outlined and illustrated by a numerical solution for the case of a mass, initially at a high uniform temperature, which loses heat from two parallel boundaries a finite distance apart. The treatment of a similar mass in which there is generation of heat is then considered and graphical solutions for these simple cases are also presented.

It is shown that a more elegant computational technique can be achieved when heat is generated by lowering the boundary temperature and, further, that the numerical work can be reduced by a simple modification of the tabular method of setting out the calculations. The utility of these devices is illustrated by application to the same example for comparison with the original solution.

The method is then extended to two and three dimensions for which the appropriate values of a non-dimensional number connecting diffusivity space and time intervals are established. To illustrate the power of the step-by-step calculation, the numerical

results of a simple two-dimensional problem are compared with the rigorous mathematical solution and show excellent agreement.

It is shown that the method can readily be applied to more complex cases. A solution is presented for the case of two lifts of concrete placed at different times on an underlying rock base which has a thermal capacity but no heat generation.

Concrete Materials and Practice

L. J. MURDOCK, Longmans Green and Co., New York (Edward Arnold and Co., London), 1948, 328 pp., $5.25 Reviewed by J. W. KELLY

We can always learn from the other fellow, and even those Americans who consider themselves well-versed in concrete would do well to see what our English cousins are doing. Murdock's "Concrete Materials and Practice" gives an excellent cross section of methods which are much like our own in principle but often different in detail. Even the difference in arrangement of subjects is stimulating, as our thought patterns have tended to become fixed and therefore not as progressive as they should be.

The book corresponds most nearly in scope to Bauer's "Plain Concrete" in America, although the parallelism is not close. It has chapters on properties of concrete, aggregates, cement, water, the design of concrete mixes, proportioning (batching materials for) concrete, mixing concrete, handling and placing, consolidation of concrete, curing concrete, concreting in cold weather, surface finishing of concrete, weathering of concrete surfaces, inspection and testing on the site, formwork, reinforcements, miscellaneous types of construction, precast concrete products, lightweight concrete, and concrete floor surfacings. Appendixes include probable variation, specific gravity and moisture content—pycnometer method, absorption and moisture content—drying methods, and method of preparation of test cubes.

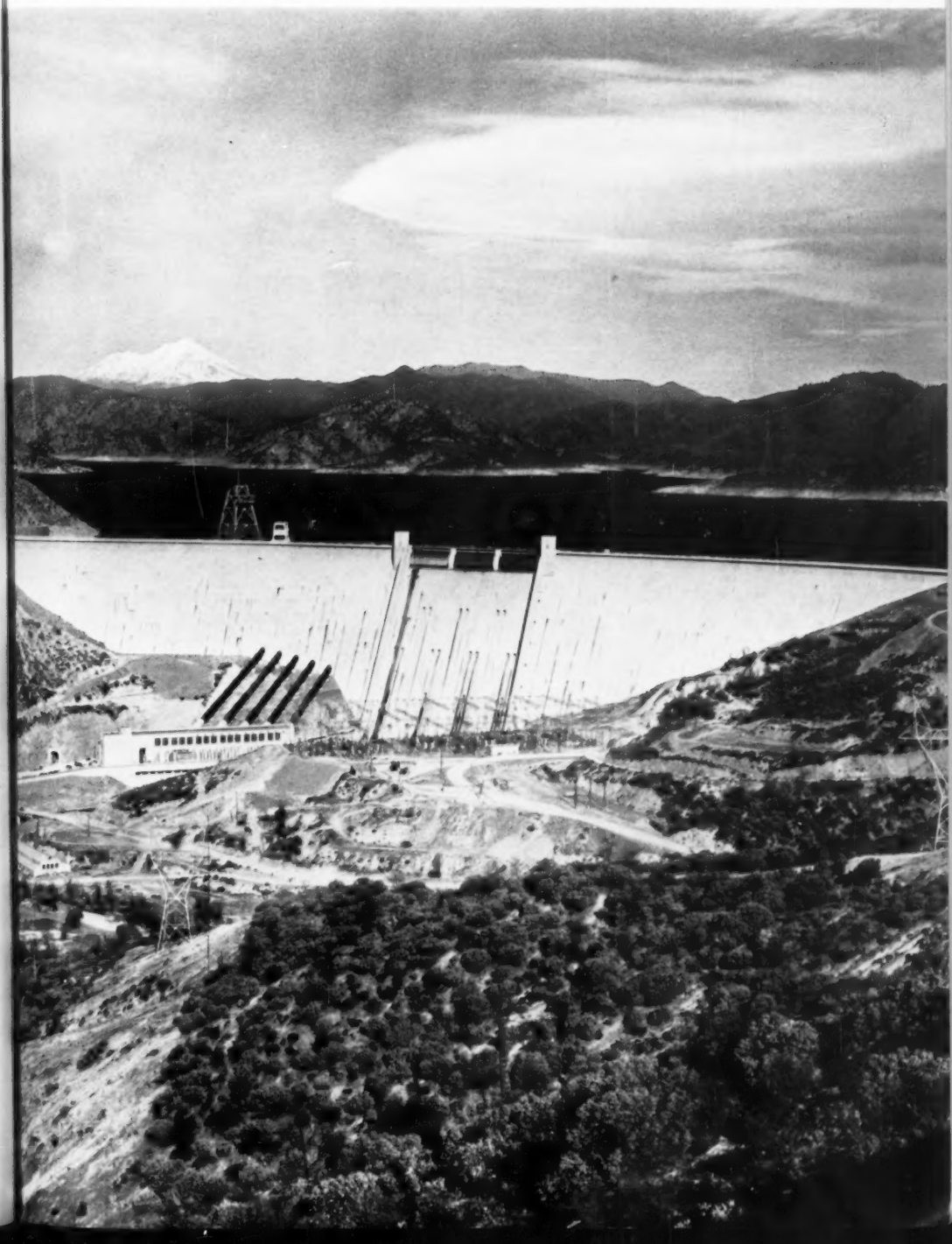
Apparently the book is written primarily for the field man such as the inspector or foreman without formal technical training. As stated by the author, "The object of this book is to provide a broad outline of the science of concrete making, particular attention being given to the translation of the knowledge gained by testing and research into practice in the field." This breadth is evident in the first two chapters which give an over-all summary including the history, properties and limitations, selection of materials, variability of quality, testing, design and its interpretation on the site, modifications in normal constructional procedure, workmanship, and properties of hardened concrete. Also the chapter on reinforcement covers briefly the principles of reinforced concrete design, control of cracking, types of reinforcement, prestressed concrete (one paragraph), working drawings, bending and placing reinforcement, code requirements, welding, and inspection.

To cover so inclusive a field in 328 pages necessarily means that many of the topics are scarcely more than mentioned; for example, ready-mixed concrete is discussed in ten lines. It does not follow, however, that the text is not thorough on those subjects which the author considers worthy of greater emphasis. In general, he has included more on specialized methods, and less on general methods, than is customary here. He lists the advantages and disadvantages of many of these methods.

Some interesting features of British practice are as follows: Mix design is based on charts such as those developed by the Road Research Laboratory. In addition to the slump test, there are a compacting-factor test and a segregation test which involve dropping the concrete through inverted cones similar to slump cones. Test cubes are used instead of cylinders. The sack of cement contains 112 lb, and the Imperial gallon contains 10 lb of water. The saturated surface-dry condition of sand is sometimes determined as the point at which, during drying, the sand first fails to stick to a glass rod. The continuous type of mixer is permitted. The author appears to be fairly familiar with American practice.

ACI News Letter

Vol. 20 No. 9 JOURNAL of the AMERICAN CONCRETE INSTITUTE May 1949



Sketch of photo of earth sample taken from mine shaft driven through Intrusion-stabilized zone.

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ACI News Letter Contents

May 1949

On the Cover—Shasta Dam and reservoir with Mt. Shasta in the distance. Key structure in the U. S. Bureau of Reclamation Central Valley Project in California, this dam—world's second highest—is 602 ft high, has a crest length of two-thirds of a mile, and contains 6 million cu yd of concrete.

Planning Chicago Convention 3

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Technical Activities Committee Planning Chicago Convention

Plans are forming for the 1950 convention in Chicago, week of Feb. 19—on the basis of a tentative schedule of seven sessions in five convention periods with two periods of two concurrent sessions each for different interest areas, and three periods of general sessions.

ACI technical committees will be largely responsible for activating brief papers and reports bearing on the work of the sponsoring committees.

These ten program session possibilities were outlined at a recent meeting; from them TAC proposes to select those seven which develop first and most promisingly:

1) The Inspection Manual—discussion of proposed revisions
2) The Building Code
3) and 4) Two sessions on general reinforced concrete design problems
5) Research—the annual open meeting of Committee 115
6) Concrete pavement problems—both design and construction
7) Concrete construction problems—possibly as a panel discussion
8) Dynamics of concrete structures—dynamic and sonic testing and related subjects
9) Plain concrete
10) Concrete materials and proportioning.

All Members are urged to express their preferences and give suggestions of sources of *desirable* subject material.

Who's Who This Month

Ralph W. Kluge, Morris M. Sparks and Edward C. Tuma

The results of Bureau of Standards tests of lightweight aggregates for the Housing and Home Finance Agency are reported by Messrs. Kluge, Sparks and Tuma in "Lightweight-Aggregate Concrete," p. 625.

Ralph W. Kluge is associate professor of structural engineering in immediate charge of instruction and research in reinforced concrete, School of Engineering, Purdue Univ. He served as structural engineer, Division of Building Technology, National Bureau of Standards from 1941-48. During the latter part of this period the reported tests were conducted. From 1929-41, Mr. Kluge was on the research staff of the Engineering Experiment Station, Univ. of Illinois. He was co-author of "Tests of Rigid Frame Bridges"—Wason Medal winner for the "most meritorious paper" in 1938. "Impact Resistance of Reinforced Concrete Slabs," Apr. 1943, JOURNAL, and "Lapped Bar Splices in Concrete Beams," Sept. 1945, JOURNAL were other contributions to the concrete field.

Mr. Sparks is hydraulic engineer for the Beach Erosion Board, Corps of Engineers, Department of the Army. He was materials engineer at the National Bureau of Standards at the time of the reported tests. In World War II he served three years in the Southwest Pacific in command of a signal assault group and later as officer in charge of a communications center, Tokyo, Japan. For his outstanding service he was awarded the Bronze Star Medal.

Mr. Tuma has been engaged in concrete research at the National Bureau of Standards since 1943. He was co-author of the ACI paper "Lapped Bar Splices in Concrete Beams." Previous engineering experience was with Kansas City Bridge and Construction Co. and the Rocky Mountain Drilling Co.

Gerald A. Leonards

In "Analysis of Building Frames with Unsymmetrical Differential Settlement of the Foundations," p. 645, Gerald A. Leonards, instructor in soil mechanics, Purdue Univ., proposes a new design approach to considering uneven settlement of footings.

From 1944 to 1946, Mr. Leonards was soils engineer for the Dept. of Transport, Canada, engaged in investigations of the design and evaluation of airport pavements. For G. L. Wiggs & Co., Consulting Engineers, Montreal, he analyzed stresses in roof trusses supporting crane rails in 1943 and 1944.

Mr. Leonards is a 1943 graduate of McGill Univ. in civil engineering and received his M.S. in C. E. at Purdue in 1948. He is a member of the Engineering Institute of Canada, Corporation Professional Engineers, Quebec, A.S.C.E. and Sigma Xi.

E. W. Scripture, Jr. and Frank J. Litwinowicz

E. W. Scripture, Jr. and Frank J. Litwinowicz describe further studies of factors affecting air entrainment in "Effects of Mixing Time, Size of Batch and Brand of Cement on Air Entrainment," p. 653. Dr. Scripture is vice president in charge of research, Master Builders Co., Cleveland and an ACI member since 1931. Previous papers on air entrainment of which he was co-author appeared in the Nov. 1948 and Feb. 1949 issues of the JOURNAL.

Mr. Litwinowicz is senior research assistant in charge of the physical testing section in the Master Builders Co. research laboratory where he has been employed since 1946. He was co-author of "Some Factors Affecting Air Entrainment," published in the Feb. 1949 JOURNAL.

Robert P. Vellines and Thomas Ason

"A Method for .Determining the Air Content of Fresh and Hardened Concrete," p. 665, reports the work of Robert P. Vellines and Thomas Ason on new apparatus and technique for this purpose.

Mr. Vellines is assistant to manager, Research Laboratories, Universal Atlas Cement Co., Buffington, Ind. After joining the firm in 1936 he was assistant section leader in the concrete laboratory and later section leader in the experimental plant. He actively participated in the development of air-entraining concrete and in the design of methods and equipment for testing it. Mr. Vellines is a 1935 graduate of W. Virginia Univ. in chemical engineering.

Thomas Ason began work with the Universal company in the physical testing laboratory in 1924 and transferred to the research laboratories in 1934. He has worked extensively on the development of physical testing methods and the standardization of physical testing procedures in the company laboratories.

Jacob Feld

Jacob Feld, consulting engineer, reports on a field experience with construction materials in "Study of Dusty Concrete Ceilings," p. 673. Recent work of Dr. Feld includes the design of concrete frames for Todt Hill and Lexington Houses in New York and the Quebec St. and Rock Creek Apartments in Washington. He was construction consultant for Veterans Hospitals at Crugers, N. Y. and Wilkes Barre, Pa. and consulting engineer on the Postwar Highway and Grade Crossing Elimination Program of the Borough of the Bronx, New York City.

Dr. Feld, a native of Austria, studied at the College of the City of New York and the Univ. of Cincinnati, and has been a consulting engineer since 1925. He is a member of Phi Beta Kappa, Sigma Xi and Sigma Tau Phi as well as A.S.C.E., American Assn. for the Advancement of Science, Brooklyn Engineers Club, and the New York Academy of Sciences. He is also a member of Department Committee on Soils, Highway Research Board, and since 1944 a member of ACI.

Sanford E. Thompson

Sanford E. Thompson, pioneer in concrete and industrial engineering, died February 25, 1949 at the age of 82.

A pioneer member of ACI, he was vice president from 1917-19 and was once chairman of the Committee on Concrete Aggregates. He was elected an honorary member of the Institute in 1932.

Mr. Thompson was first chairman of A.S.T.M. Committee C-9 on Concrete Aggregates and served in that capacity from 1914 through 1922. In recognition of his work, the Sanford E. Thompson Award was established and he was made an honorary member of that committee.

In World War I he served as a Lt. Col. and was a full-time consultant to the Secretary of War on ordnance production in World War II.

In later years, until his retirement in January 1949, he devoted his time to industrial management as a member of the firm of Thompson & Lichtner, Brookline, Mass.

J. Morley Zander

J. Morley Zander, general manager of the Saginaw Brick Co., died January 11, 1949 at Saginaw, Michigan. He was a registered professional engineer and an authority on the manufacture of sand-lime and concrete brick and the installation of brick-making machinery. He served as president and later as secretary-treasurer of the Sand-Lime Brick Assn.

William R. Kleckner

Word has just been received of the death of William R. Kleckner on Sept. 2, 1948. Mr. Kleckner was works manager and chief chemist of the Pacific Portland Cement Co., San Francisco, California, with which he had been associated for 27 years. Mr. Kleckner had been an ACI member since 1944.

New Members

With 122 new Member applicants approved for the month of March the new membership total became 4667 as of March 31, 1949.

The geographic distribution of new Members shows Michigan and California the two top member-producing states with eleven applicants each, New York close behind with ten applicants.

California

Balagot, Porfirio, (Jr.) Holmes & Narver, Okinawa Engineer District, APO 331, c/o P. M., San Francisco, Calif.
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Flick, Robert J., (St.) 4429 Cutting Blvd., Apt. 2C, Richmond, Calif.
Gill, S. S., (St.) 450 International House, Berkeley 4, Calif.
Hare, Jack Allison, (St.) 702 62nd St., Oakland 9, Calif.
Judd, Edmund H., (Indiv.) P.O. Box 668, San Carlos, Calif.
McLean, Ralph S., (Indiv.) 309 Jergins Trust Bldg., Long Beach 2, Calif.
Pestana, Charles Haven, (Indiv.) 3526 Custer St., Oakland 1, Calif.
Showell, Carter S., (Indiv.) c/o U. S. Bureau of Reclamation, Orange Cove, Calif.
Sison, Pedro Y., (Jr.) Guam District, Corps of Engineers, APO 246, c/o P.M., San Francisco, Calif.
Thompson, Paul D., (Indiv.) P.O. Box 985, Modoc County, Tioneste, Calif.

Colorado

Spratlen, F. P., Jr., (Indiv.) Ready Mixed Concrete Co., 1800 E. 46th Ave., Denver 16, Colo.

District of Columbia

Sparks, Morris M., (Indiv.) 2231 California St., N. W., Washington 8, D. C.

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Turner, C. H., (Indiv.) P.O. Box 544, Waycross, Ga.

Illinois

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Gray, W. B., (Indiv.) c/o Holabird & Root & Burgee, 180 N. Wabash Ave., Chicago 1, Ill.
Lyon, Charles A., (Indiv.) c/o Master Builders Co., 228 N. LaSalle St., Chicago 1, Ill.

Indiana

Garcia, Alberto A., (St.) 371 Dillon Hall, University of Notre Dame, Notre Dame, Ind.
Montiel G., Jose Luis, (St.) 530 N. Allen St., South Bend, Ind.

Iowa

Tooles, Calvin Warren, (Jr.) 1203 Wilson Ave., Ames, Ia.

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Louisiana

Jahncke, Herbert, (Indiv.) 814 Howard Ave., New Orleans 4, La.
Murry, John D., (St.) P.O. Box 212, Ruston, La.

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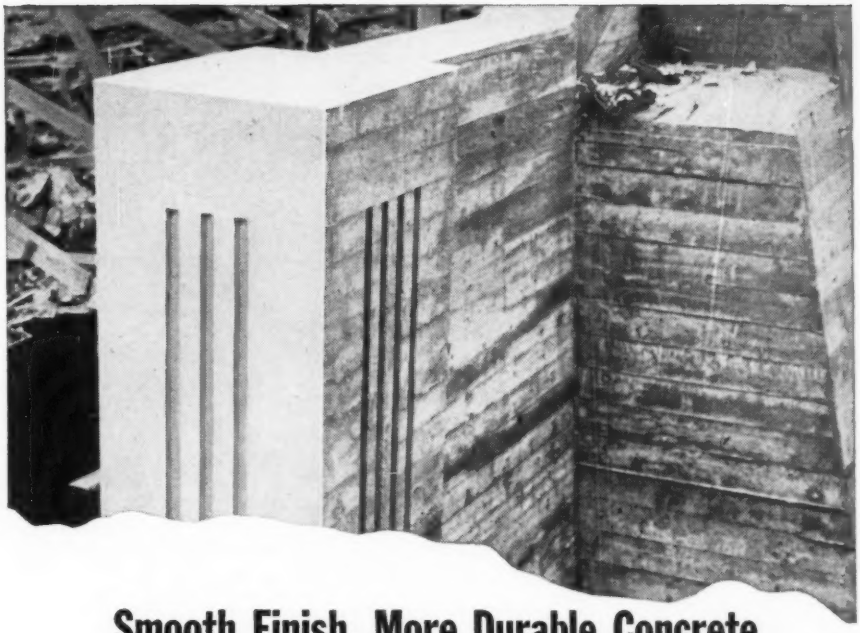
Ferrer, Jesus R. Jr., (St.) 417 E. Liberty, Ann Arbor, Mich.
Gonzalez, Felix M., (St.) 417 E. Liberty St., Ann Arbor, Mich.
Hsu, Kuo-liang, (St.) 1358 Geddes Ave., Ann Arbor, Mich.
Hu, Lu-shien, (St.) 1345 Geddes Ave., Ann Arbor, Mich.
Irani, Rustom Ardeshir, (St.) c/o International Center, 613 E. Madison St., Ann Arbor, Mich.
Kolm, Alwin S., (Indiv.) 22 W. 25th St., Holland, Mich.
Legg, Frank E. Jr., (Indiv.) 1224 E. Engr. Bldg., University of Michigan, Ann Arbor, Mich.
Lluch, Roberto S., (St.) 633 Church St. Ann Arbor, Mich.
Yu, Herbert, (St.) 417 E. Liberty, Ann Arbor, Mich.

Minnesota

Bunea, S. Paul, (Indiv.) 1954 Grand Ave., Apt. 205, St. Paul 5, Minn.



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Murray, Victor F., (Indiv.) Box 247, Hoboken, N. J.

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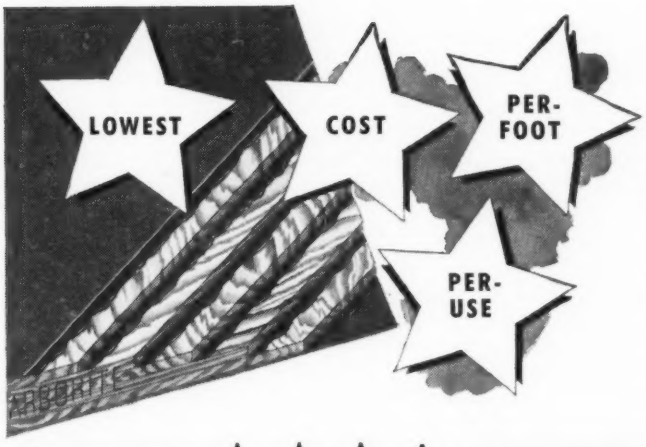

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Triscari, Frank Paul, (St.) 107 Arnold St., Laramie, Wyo.
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Yamada, Kaoru, (Indiv.) P. O. Box 509, Paia, Maui, T. H.

Australia

Langlands, Ian, (Indiv.) Council for Scientific & Industrial Research, Building Research, Graham Rd., Highett, S. 21, Victoria, Australia

Canada

Beam, Howard A., (Jr.) R. R. No. 1, Ridgeway, Ont., Canada
Chernick, Alexander, (Indiv.) 37 Hilton Ave., Toronto 10, Ont., Canada
Dillon, M. Murray, (Indiv.) 365 Richmond St., London, Ont., Canada
Donaldson, William A., (Indiv.) 11133 71 St., Edmonton, Alberta, Canada
Goodman, F. I. C., (Indiv.) Asst. Engr., Chief Engineer's Office, C. P. Ry., Rm. 401, Windsor St. Station, Montreal, Que., Canada
Hendrick, Gerald J., (Indiv.) Box 79, Renfrew, Ont., Canada
Keith, Grant Turner, (St.) 808 29th St. W., Saskatoon, Sask., Canada
Royer, Maurice, (Indiv.) 239 Laurier Ave., Quebec, Que., Canada

Costa Rica

Moya A., Rafael, (St.) Avenida 16, Calles 5 y 7, Casa Numero 504, San Jose, Costa Rica

Cuba

Docal, Abelardo L., (Indiv.) 27th St., No. 902, Vedada, Havana, Cuba
Galbis M., Ricardo, (Indiv.) Calle 64 casi esquina 5a Ave., Miramar, Havana, Cuba
Garcia, Ignacio, (St.) Ave. de America 153, Alt. de Almendares, Havana, Cuba
Gonzalez N., Braulio, (Indiv.) Santa Rosa 69, entre Castillo y Fernandina, Cerro, Havana, Cuba
Gonzalez P., Jose Antonio, (Indiv.) Calle Sola No. 117, entre Luis Estevez Y Estrada Palma, Santos Suarez, La Havana, Cuba
Llarena S., Vicente, (Indiv.) Ave. 11 entre calles 11 y 12, Ampliacion de Almendares, La Havana, Cuba
Mendez-Pesant A., Alfonso, (Indiv.) Calle 10 No. 519 entre 21 y 23, Vedado, Havana, Cuba
Perez de Morales M., Eduardo, (Indiv.) Jovellar 315, 3° piso, Havana, Cuba

Czechoslovakia

Beran, Jaroslav K., (Indiv.) Faculty of Civil Engineering, University of Technical Sciences, Karlovo-nam, 17, Prague 2, Czechoslovakia

Egypt

Fouad, El-Sayed Whley El-Din, (Jr.) 19 Fatma Nabawieh St., Abbassieh, Cairo, Egypt

El Salvador

Duarte, Victor Manuel, (St.) Calle Ramon Belloso No. 18, San Salvador, El Salvador, C. A.

Finland

Hanson, K. W., (Indiv.) Bredviksvagen 1, Munksnas, Helsingfors, Finland

Guatemala

Hegel Andreu, Guillermo, (St.) 55 C. O. No. 3, Guatemala City, Guatemala, C. A.

India

Chaturvedi, R. S., (Indiv.) Irrigation Research Officer, Roorkee, (U. P.), India
Mahimtura, Ramanlal Harilal, (Jr.) "Shree Ramnivas", 7th Khetwadi Lane, Bombay 4, India

Continued on p. 16

Positions and Projects — ACI Members

All Star Nominating Committee

The 1949 Nominating Committee seems to constitute such an all star cast that it deserves special mention. The By-laws provide that each year's Nominating Committee shall consist of five elected members chosen from among 20 candidates named by the Nominating Committee of the year before, plus the three ACI presidents last past. The tally reported by the tellers on the 1948 election on the second day of the 1949 convention showed Franklin R. McMillan as receiving the most votes–thus chairman of the Nominating Committee. The four other elected members of the Nominating Committee are also past presidents of the Institute: Raymond E. Davis, Roy W. Crum, Frank E. Richart and Morton O. Withey. These five will join with Robert F. Blanks, Stanton Walker and Harrison F. Gonnerman, the three presidents last past as required in the By-laws. If there is anything in long and close association with the administrative work of the Institute which would fit a man especially for the responsibility of the Nominating Committee, it is probably right to say that the 1949 Nominating Committee is all star in its composition.

British Cement and Concrete Assn. expands research in concrete

The Research and Development Division of the Cement and Concrete Assn. (England) has put into operation a research station at Wexham Springs near Stoke Poges, Buckinghamshire. W. A. Fairhurst, technical director, has general control of the research program. He is assisted by a Research Advisory Committee which includes A. L. L. Baker, professor of concrete technology at the Imperial College of Science, London Univ.

Chemical and concrete testing laboratories are already in use and facilities for research in concrete structures and precast concrete products are planned. The four main parts of the research division are chemical, concrete and concrete materials, structures and development.

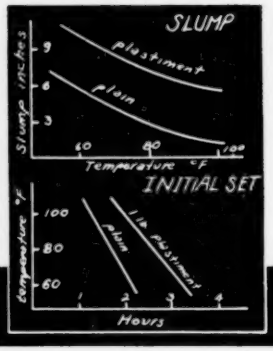
The first problems to be studied are durability of concrete, effects of types and proportions of aggregates and mix design on the properties of concrete, methods of curing to obtain high early strength and surface finishes for architectural concrete and pavements. The effect of air entrainment on durability and whether cement-aggregate reaction is a factor in Great Britain are also being investigated.

Personnel changes in N.S.G.A. and N.R.M.C.A.

Fred F. Bartel has resigned as Assistant Director of Engineering of the National Sand and Gravel Assn. and as Assistant to the Director of Engineering of the National Ready Mixed Concrete Assn. to accept the position of Engineer for the Tews Lime and Cement Co. of Milwaukee, Wisconsin. During his service with the Assns. Mr. Bartel was active in committee work of A.S.T.M., ACI and the Highway Research Board. He was assistant secretary of A.S.T.M. Committee C-9 on Concrete and Concrete Aggregates.

Mr. Bartel is succeeded by Delmar L. Bloem who, until his promotion, was Associate Research Engineer. Mr. Bloem joined the Assns.' staff in 1945 and has been actively in charge of their research activities at the Univ. of Maryland. He is the author of several papers dealing with concrete and concrete aggregates published by the Assns., ACI and A.S.T.M.

E. J. Zeigler replaces Mr. Bloem as Associate Research Engineer in direct charge of laboratory activities at the Univ. He served as a part-time student-assistant in the Assns.' laboratory while attending the Univ. of Maryland and joined the full-time staff as Assistant Research Engineer in 1946.



HOT WEATHER CONCRETE

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Iowa Building Code Council reports

The Iowa Building Code Council, Frank Kerekes, chairman, recently reported a proposed "Iowa State Building Code" to the governor. It is being considered for adoption by the State General Assembly.

Groundwork for the code began with a summary and analysis of building legislation affecting the design, construction and maintenance of buildings in 19 cities in Iowa. In addition to the technical requirements of building legislation and construction, the principal criticisms, faults and shortcomings of existing building codes in other states and cities were carefully analyzed to minimize or eliminate similar defects in the Iowa code.

Legislation proposed by the Council would establish a uniform statewide building code providing minimum standards and specifications for construction throughout the state but permitting cities and towns to prescribe by ordinance higher standards when necessary.

The legislative act would establish general standards for construction, fire protection, health and sanitation and safety while detailed specifications would be rules and regulations of the state building code council, permitting changes to meet new and changing conditions, new types of construction and materials and new building design.

Reinforced concrete construction is specified to be in accordance with ACI "Building Code Requirements for Reinforced Concrete."

New Committee 318 secretary

J. P. Thompson, Structural and Railways Bureau, PCA, long interested and active in the work of Committee 318-Standard Building Code, has been appointed secretary of the committee. The April News Letter announced the appointment of Chester L. Post as chairman.

Reviewers needed

To broaden the scope of the Current Reviews section of the JOURNAL, reviewers of foreign-language technical publications —Spanish and Italian, particularly—are needed. Members who wish to volunteer their services are urged to write the Secretary stating language specialty and fields of interest. It may be that publications presently received cannot be fitted to individual interests. In those cases which indicate a gap in review coverage, appropriate periodicals will be looked for. Duplication of offers in a given language will provide a reserve against the loss of those now serving.

Barton enters merchandising field

J. Frank Barton, chief chemist and research engineer, Federal Portland Cement Co., Inc., Buffalo, N. Y., has resigned to enter the merchandising field with the Household Art Co., Hornell, N. Y. Mr. Barton received his B. S. in

chemistry and geology at the Univ. of Maryland. He started his career in the cement industry in 1925 as analyst for the New Egyptian Portland Cement Co., Port Huron, Mich. He gave considerable time to research with pozzolana cements and to developing manufacturing control of home and industrial insulation. He also developed an artificial pozzolanic material for use in concrete, a general purpose masonry cement, a high temperature insulating cement and patented a dry-chemical feeder.

Concreting process studied

A study of the Prepakt method of making concrete is under way at the Clinton, Miss., suboffice of the Waterways Experiment Station. Originally developed to make repairs to deteriorated concrete structures, the use of the process in new construction is being investigated. The test involves the construction of two models, an 85-cu yd dam section containing galleries and tunnels and a 15-cu yd model of a reinforced concrete bridge pier. The models will be extensively tested for strength and durability.

This work is a part of an extensive program of civil works investigations by the Office, Chief of Engineers. The study is under the supervision of Herbert K. Cook, Chief of the Concrete Research Div., Waterways Experiment Station. Thomas P. Kennedy is project engineer and J. H. Bowman represents Intrusion-Prepakt, Inc. All are Members of ACI.

Calaveras president honored

William Wallace Mein, Jr. president, Calaveras Cement Co., San Francisco, Calif., was presented a gold watchfob by Dr. W. E. Wrather, outgoing president of the A.I.M.E., at the recent meeting in San Francisco, on behalf of the members of the San Francisco section, in recognition of his work as program chairman of the section during the past year. Mr. Mein also served as associate chairman of a session on mineral aggregates and structural materials, sharing honors with Dr. Oliver Bowles of the U. S. Bureau of Mines, one of the nation's foremost experts on the technology and economics of industrial minerals.

Brunnier goes to Japan

H. J. Brunnier, consulting structural engineer of San Francisco, has gone to Japan, at the request of the American Assn. of Seismology and with the cooperation of the Army, to investigate the effects of the Fukuii earthquake.

New engineering practice

William J. Wenzel recently resigned the post of director of public works, Great Falls, Mont., which he had held since 1947, to establish a private engineering practice in Great Falls. Many city improvements were made during his tenure as director of public works.

Annual Report of Engineers' Council for Professional Development

The 16th annual report of the Engineers' Council for Professional Development, a conference of engineering bodies organized to enhance the professional status of the engineer, reviews the work of the council in the form of reports by its several committees.

The four major committees are Student Selection and Guidance, Engineering Schools, Professional Training, and Professional Recognition with the apparent aim of assuring the selection of individuals best suited to engineering training, then providing adequate technical training and finally working toward better recognition of the professional status of the engineer.

In addition, the annual report lists accredited undergraduate engineering curricula in the United States by institutions and by fields of engineering. Also included are accredited programs of technical institute type in the broad area of technical education between professional engineering and vocational education.

Honor Roll

February 1, 1949 to March 31, 1949

Prof. Newlin D. Morgan, University of Illinois, continues to be top man on our Honor Roll for 1949 with credit for 42½ new Member applicants.

Prof. F. N. Menefee, University of Michigan, enters the race in second place with nine credits and Jaime Alberto Mitrani, Havana, Cuba, is in third place with six credits.

Newlin D. Morgan	**42½**
F. N. Menefee	**9**
Jaime Alberto Mitrani	**6**
Newlin D. Morgan, Jr.	**5**
J. Vicente Orozco	**4**
Jose Antonio Vila	**4**
H. D. Jolley	**3**
Stanton Walker	**3**
Roger H. Corbetta	**2½**
Raymond E. Davis	**2**
John J. Hogan	**2**
Emory E. Johnson	**2**
Karl W. Lemcke	**2**
A. J. McElrath	**2**
William T. Neelands	**2**
William D. Nowlin	**2**
Leon Venegas	**2**
Roberto Barrilas F.	**1½**
R. A. Caughey	**1½**
H. J. Gilkey	**1½**
H. F. Gonnerman	**1½**
D. H. Pletta	**1½**
Paul W. Abeles	1
William F. Anderson	1
Samuel Baker	1
Charles F. Ball	1
John H. Bateman	1
J. F. Baxter	1
Oscar Benedetti	1
Robert F. Blanks	1
Hans H. Bleich	1
Carlos Mora Bravo	1
Milton Brumer	1
S. J. Chamberlin	1
Alirio Cornejo	1
W. S. Cottingham	1
Ray A. Crysler	1
Peter J. Doanides	1
A. T. Docal	1
W. H. Droege	1
Belmon U. Duvall	1
J. M. Faircloth	1
F. L. Fitzpatrick	1
Robert W. Freeman	1
Armour T. Granger	1
Murray E. Griset	1
J. W. Hubler	1
J. P. Jain	1
V. P. Jensen	1
Barton Jones	1
R. R. Kaufman	1
J. W. Kelly	1
K. V. Kini	1
George E. Large	1
Leslie D. Long	1
Glenway Maxon	1
James A. McCarthy	1
M. J. McMillan	1
Carl A. Menzel	1
Jesus Mesa	1
Robert B. B. Moorman	1
Robert Morris	1
George G. Nichols	1
LeRoy T. Oehler	1
Emil Praeger	1
Raul Miguel Rivero	1
Edward W. Scripture, Jr.	1
Roy T. Sessums	1
Saul Shaw	1
Thomas C. Shedd	1
Richard Skalak	1
Luis G. Solano A.	1
Alois Starkmann	1
Manabu Tanaka	1
Maurice Taylor	1
T. W. Thomas	1
G. E. Troxell	1
Oscar J. Vago	1
Joseph J. Waddell	1
Stanton Walker	1
John M. Wells	1
Frank W. Wheeler	1
Donald K. Woodin	1

The following credits are, in each instance "50-50" with another member.

Jerome O. Ackerman	Phil M. Ferguson
C. Harold Barcus	Ambrosio R. Flores
Charles H. Barker	Benjamin Forsyth
Frank Baron	Adly W. Gindy
E. W. Bauman	A. W. Haddow
R. W. Brandley	J. R. G. Hanlon
A. H. Brewer	Adli Milik Hanna
Harold W. Brewer	M. J. Hawkins
A. R. Brickler	Elmo C. Higginson
R. Y. Bush	Albert C. Hooke
J. A. Carrick	Bruce Jameyson
Napoleon V. Campomanes	E. R. Kendall
	Max Krueger
Harold S. Carter	Henri Marcus
Arthur P. Clark	Hudson Matlock
R. D. Conrow	J. W. McBurney
Lauro M. Cruz	T. R. McCullough
W. A. Davis	Jack Robert Miller
Harry Delzell	William S. Moore
A. H. Douglas	Wm. Morris
W. J. Emmons	Philip M. Noble
J. L. Faisant	George L. Otterson

Continued on p. 16

"BUY U. S. SAVINGS BONDS DURING THE OPPORTUNITY DRIVE,"

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WILLIAM GREEN, ***President, American Federation of Labor***

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PHILIP MURRAY, ***President, Congress of Industrial Organizations***

"The C.I.O. has endorsed every effort to encourage the worker to put more of his earnings into U. S. Savings Bonds. They represent both security and independence."

WINTHROP W. ALDRICH, ***Chairman, Chase National Bank***

"I believe that every individual who can possibly do so should buy more U. S. Savings Bonds. These bonds represent one of the best investments of our time."

CHARLES F. BRANNAN
Secretary of Agriculture

"I am heartily in favor of the Opportunity Drive to buy more U. S. Savings Bonds. Everyone engaged in farming should recognize the importance of a backlog of invested savings as a means of realizing the agricultural opportunities of the future."

DURING MAY AND JUNE, the U. S. Savings Bond Opportunity Drive is on! It is called the Opportunity Drive—because it is truly an opportunity for *you* to get ahead by increasing your own personal measure of financial security and independence.

If you haven't been buying Savings Bonds regularly, start *now*.

If you have been buying them, add an *extra* Bond or two to your purchases this month and next. Remember—you'll get back $4 for every $3 in a short ten years' time!

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New Members

Continued from p. 10

Israel

Gruenwald, Ernst, (Indiv.) 10, Jerusalem-street, Haifa, Israel

Mexico

Garza, Ignacio, (Indiv.) Heriberto Frias 623, Col. Narvarte, Mexico D. F., Mexico

Ingenieros Civiles Asociados S. A. de C. V., (Corp.) Heriberto Frias 623, Mexico D. F., Mexico (Bernardo Quintana)

Pescador, Felipe, (Indiv.) Heriberto Frias No. 623, Col. Narvarte, Mexico D. F., Mexico

Ramirez, Sabas Gonzalez, (Jr.) Pino Suarez No. 226, Leon, Guanajuato, Mexico

Zamora M., Francisco, (Indiv.) Romero de Terreros No. 1405, Mexico D. F., Mexico

New Zealand

Palmer, James Maurice, (Indiv.) Box 68, Dunedin, Otago, New Zealand

Turkey

Bernhard, Kurt, (Indiv.) 69 Ismet Pasa Caddesi, Yenisehir, Ankara, Turkey

Venezuela

McGee, James Herman Taylor, (Indiv.) Sur 17 Bis No. 18 "El Conde", Caracas, Venezuela

Honor Roll

Continued from p. 14

Chester B. Palmer
Douglas E. Parsons
Jose F. Perez
Robert J. Perrich
John A. Ruhling
Fred H. Ryan
R. H. Sherlock
Jack S. Short
Ruben Ruiz Silva
H. F. Sommerscheild
G. W. Stokes
E. O. Sweetser
Anton Tedesko
J. Trueman Thompson
LeRoy A. Thorssen
Homer Thrall
Fehiman Tokluoglu
John Tucker, Jr.
I. L. Tyler
F. E. Votaw
W. Wayne Wallace
M. H. Walters
David Watstein
J. C. Watt
W. R. Waugh
Carl Weber
Charles S. Whitney
H. F. Williams
Paul C. Winters
Thomas Worcester
K. W Wright
Alexander H. Yeates
Robert Zaborowski

Tools, Materials, Services

Under this heading note will be made from time to time of producer literature of presumed technical interest (and available from its source for the asking) to ACI users of tools, equipment, materials, accessories and special services.

Careers in engineering and science

Representing a departure from the typical college catalogue this new career reference book published by the Polytechnic Institute of Brooklyn gives a simple and direct presentation of the opportunities available for professional training in an engineering college. Its main consideration is the point of view of the high school student and the adult leaders who work with young men.

What should interest young men planning to become engineers and the lay public in general is the review of the tremendous battery of scientific instruments and facilities available in today's engineering college. From this standpoint, the book might be considered a survey of modern science as it applies to engineering.

Throughout the booklet an effort has been made to give an intimate picture of the everyday activity in the classrooms and laboratories of a typical engineering college. While the illustrations were of necessity drawn from activities at the Polytechnic, the point of view has been kept general so that this publication may be an effective guidance booklet no matter what college a student may consider.

For further information, write Prof. Cornelius Wandmacher or Mrs. Louise Warner, publicity director for the Polytechnic Institute of Brooklyn.

Admixture claimed to reduce water absorption by concrete

Hydropel is an aqueous suspension of colloidal asphalt developed for use in portland cement concrete and mortars. Added at the usual rate of 1½ gal. per sack of cement, it is said to be effective as an integral admix to reduce water absorption by cement concrete and mortars, with subsequent reduction in expansion and contraction, and no serious impairment of strength. Improved workability and dispersion of cement, reduction in water content, increased resistance of concrete to abrasion, and increased ability of the concrete to flex and absorb shock without breaking are attributed to the material.

Hydropel is a product of the American Bitumuls Co., 200 Bush St., San Francisco 4, Calif.

Hydraulic weighing cells

A new 8-page bulletin describes and illustrates Emery hydraulic cells for general industrial weighing and special laboratory testing equipment. The bulletin also describes three self-contained, portable weighing units or scales with Bourdon tube indicators and several cell applications with the Taite-Emery indicator.

Copies of Bulletin No. 288 may be obtained from the Baldwin Locomotive Works, Testing Equipment Dept., Philadelphia 42, Pa.

SYNOPSES of recent ACI Papers and Reports

Institute papers of this JOURNAL Volume 20 are currently available at prices indicated. Please order by title and title number.

RECOMMENDED PRACTICE FOR WINTER CONCRETING METHODS (ACI 604-48).....45-1

Price 50 cents.

REPORT of COMMITTEE 604—Sept. 1948, pp. 1-20, (V. 45) in special covers

Supersedes 44-13.

This ACI standard establishes methods of cold-weather concreting for thin sections and mass concrete. Heating of materials, accelerators and anti-freezes, curing and temperature records during curing, subgrade (or base) preparation, protective coverings during curing, and form removal are discussed for both types of job, and preferred methods are indicated. An appendix entry outlines objectives of the special winter methods with background material which indicates the "why" of some of the recommended practices. Charts in the appendix indicate effect of curing temperature on concrete strength, and a list of 135 selected references to periodical literature on winter concreting methods is included.

RELATIVE STRENGTHS OF PORTLAND CEMENT MORTAR IN BENDING UNDER VARIOUS LOADING CONDITIONS.....45-2

Price 35 cents.

JACOB PORTER FRANKEL—Sept. 1948, pp. 21-32 (V. 45)

Assuming the behavior of standard mortar to be similar to that of plain concrete, tests were performed on 99 small mortar beams under sixth-, third- and center-point loading to verify the applicability of the statistical theory of the strength of brittle materials to concrete structures. The theory, as first developed by Weibull, is briefly analyzed, compared to the methods used by Tucker, and finally applied to the tests at hand. Agreement between the theoretical and experimental findings is so close as to warrant future testing on plain concrete specimens.

TERRAZZO AS AFFECTED BY CLEANING MATERIALS.....45-3

Price 35 cents.

D. W. KESSLER—Sept. 1948, pp. 33-40 (V. 45)

Effect of cleaning materials on terrazzo was studied using solutions of the following detergents: soda ash, trisodium phosphate and synthetic sulfonate. Seventy-six different aggregates were used in preparing terrazzo disks which were moist cured 3 months and then surface ground to form a shallow dish. Detergent was allowed to stand in dish 30 minutes before rinsing and drying (at 105 C). This testing cycle was repeated until deterioration tendencies were established. Soda ash solution proved much more destructive than the trisodium phosphate; however, up to the time of writing (after 850 cycles) the synthetic sulfonate had caused no failure.

TRANSPORTING READY-MIXED CONCRETE IN OPEN DUMP TRUCKS.....45-4

Price 35 cents.

R. A. BURMEISTER—Sept. 1948, pp. 41-56 (V. 45).

Milwaukee experience in hauling air-entraining concrete in open dump truck bodies is recorded. Slump, air content, specific weight, compressive strength and workability tests were made on concrete before and after a trial run over bumpy pavement. Some similar tests were made on concrete being placed in a Milwaukee street after open truck transportation. The author concludes that open trucks for hauling ready-mixed air-entraining concrete are satisfactory when air content is between 3 and 6 percent (5 to 6 sacks cement per cu yd) and hauling time is 45 minutes or less, provided that aggregate moisture, grading and slump are carefully controlled.

A STUDY OF ALKALI-AGGREGATE REACTIVITY BY MEANS OF MORTAR BAR EXPANSIONS.....45-5

Price 35 cents.

T. M. KELLY, L. SCHUMAN and F. B. HORNIBROOK—Sept. 1948, pp. 57—80 (V. 45)

Many types of aggregate were combined in varying amounts and sizes with high- and low-alkali cements and formed into 1 x 1 x 10-in. mortar bars. The bars were stored either at 70 F or at 100 F and their expansions measured at ages ranging from 1 month to 4 years.

In combination with high-alkali cements, opal, opaline chert and a siliceous dolomitic limestone were found to cause greatest expansion. Certain aggregates containing volcanic glasses and some natural sands and gravel also caused excessive expansion; with one exception, these sands contained small amounts of opal.

Greatly delayed expansion resulted with the very fine sizes of opal, particularly in combination with high-soda cement. Similar behavior resulted with minus No. 81 size opal and low-alkali cement with either Na_2SO_4 or K_2SO_4 additions.

Materials such as dehydrated kaolin, soda feldspar, magnesium fluosilicate, acetic acid and calcium hydroxide added in small amounts as correctives were ineffective. However, diatomaceous earth in sufficient quantity as a cement replacement eliminated expansion.

REINFORCED CONCRETE WALL AND COLUMN FOOTINGS PART I.....45-6a

Price 60 cents.

FRANK E. RICHART—Oct. 1948, pp. 97-128 (V. 45)

PART II.....45-6b

Price 35 cents.

FRANK E. RICHART—Nov. 1948, pp. 237-260 (V. 45)

This paper presents, in two parts, a report on an experimental investigation of reinforced concrete footings. It represents the first extensive study of the subject since the well-known work of Talbot in 1913. In the meantime there have been many developments in materials, as well as in design methods.

Important developments from these tests include the following: (a) Definite proof that the tensile and bond resistance depend upon the bending moment and shear found by statics by consideration of the full applied load, and not 85 percent thereof, as assumed in current building codes; (b) Hooked ends of bars showed no particular advantage in bond resistance over straight bars, particularly when deformed bars of new improved types were used; (c) Hooked bars produced very little effect on resistance to diagonal tension, much less than is commonly assumed; (d) Welded mats proved particularly effective in resisting end slip of bars; (e) Footing caps or piers are effective in reducing the amount of reinforcement and in increasing the load capacity of footings; and (f) Diagonal tension seems to be the point of weakness in current design practice. The factor of safety of thin footings in this respect appears greater than in thick footings, and is generally greater in rectangular than in square footings when the conventional methods of computation are used.

In these tests, major emphasis has been placed on the isolated column footing. Principal attention has been

given to the resistance of footings to failure by bond, diagonal tension and tension in the steel. Test footings were designed to produce these various types of failure. In addition, studies were made of the behavior of rectangular footings and footings having intermediate capitals or piers.

The tests featured measurements of tensile and compressive strains, deflections and slippage of bars. In some of the tests three grades of concrete were used, in others, five types of reinforcing bar were employed. In most of the tests, the footings were 7 ft square. They were supported on a bed of steel springs and tested in a large testing machine.

This paper merely attempts to present and interpret the test results, and does not offer recommendations as to possible changes in design procedures at this time.

LATTICE ANALOGY IN CONCRETE DESIGN.....45-7

Price 35 cents.

DOUGLAS McHENRY—Oct. 1948, pp. 129-140 (V. 45)

The lattice analogy is a scheme for solving two-dimensional stress problems in which the involved mathematical methods of the theory of elasticity are replaced by simple computations. The solid section is replaced by an equivalent lattice or framework which may be solved by methods applicable to indeterminate structures. Stress distribution in sections of complex shape and with complex loading may be determined by successive approximations which involve only substitution in simple equations. The method is illustrated by application to the problem of stresses in a deep beam with off-center loading.

STRENGTH OF PRECAST CONCRETE FLOOR JOISTS.....45-8

Pricd 35 cents.

JACOB FELD—Oct. 1948, pp. 141-148 (V. 45)

The strength and usability of precast concrete floor joists left in the open without any protection for a year, when a housing project was abandoned, were proved by load test on a slab section incorporating the poorest joists. This paper describes the condition of the joists, the load test results and indicates that a greater tolerance can be safely permitted in the visible defects of precast concrete joists.

EFFECT OF ENTRAINED AIR ON CONCRETES MADE WITH SO-CALLED "SAND-GRAVEL" AGGREGATES.....45-9

Price 35 cents.

PAUL KLIEGER—Oct. 1948, pp. 149-164 (V. 45)

Sand-gravel aggregates (maximum particle size ⅜ in.) used with non-air-entraining cement produce concretes containing from 3 to 4 percent air. The same mixes made with air-entraining cement and the sand-gravel aggregates have air contents from 8 to 13 percent. Effect of this larger amount of entrained air is given in terms of tests of flexural and compressive strength, freezing and thawing resistance, and length changes in varying storage conditions. Effect of "sweetening" the sand-gravel by addition of 30 percent of coarser aggregate is noted.

BURNED SHALE AND EXPANDED SLAG CONCRETES WITH AND WITHOUT AIR-ENTRAINING ADMIXTURE.....45-10

Price 35 cents.

P. H. PETERSEN—Oct. 1948, pp. 165-176 (V. 45)

The physical properties of several lightweight aggregate portland cement concretes made with burned shale or expanded slag were investigated at the National Bureau of Standards. Three grades of concrete were made with each aggregate. Air-entrainment greater than 20 percent is reported for the mixtures leanest in cement, an air-entraining admixture being used to increase the workability of all but the richest concretes. Compressive, flexural and bond strength data are given as well as resistance to heat transfer, rain penetration and water penetration by capillarity. Also included are the coefficients of thermal expansion, shrinkage, and values for change in length due to wetting and drying.

CEMENSTONE PRECAST CONSTRUCTION.....45-11

Price 35 cents.

A. C. GRAFFLIN—Nov. 1948, pp. 193-204 (V. 45)

Standardized precast reinforced concrete structural members and panels point toward economy in fireproof construction of ordinary buildings up to four stories. The method described has been so developed that architects and engineers can select from tables reinforced concrete beams, columns, roof, floor, and wall panels in the same manner as for structural steel. Design is in accordance with the ACI Code and costs compare favorably with similar structures framed of steel and fireproofed with concrete.

SAND GRADING INFLUENCE ON AIR ENTRAINMENT IN CONCRETE.45-12

Price 35 cents.

M. A. CRAVEN—Nov. 1948, pp. 205-216 (V. 45)

Four series of mixes with varying air-entraining agents, cement factors, and sand grading and content were prepared in order to observe sand grading effect on concrete. Graphic record is presented of air content and W/C plotted against fineness modulus of sand, flow and compressive strength plotted against fineness modulus of sand, and air content plotted against percent of No. 30—No. 50 sand and total sand percentage. Generally the percentage of air entrained in concrete increased with decrease in fineness modulus of sand. Quantity of air appears to be a function of the quantity of No. 30—No. 50 sand. Effect of grading and quantity of sand on other properties of fresh and hardened concrete is noted.

INFLUENCE OF SIZE GRADING OF SAND ON AIR ENTRAINMENT.45-13

Price 35 cents.

E. W. SCRIPTURE, Jr., F. B. HORNIBROOK, and D. E. BRYANT—Nov. 1948, pp. 217-228 (V. 45)

Field reports indicated difficulty with certain sands in securing the desired amount of entrained air, this was frequently attributed to the size grading of the sand, particularly to a deficiency in the finer fractions. An experimental investigation was undertaken in view of the paucity of published data on this subject. Mixes of sand and water alone, 1:4 and 1:2 mortars, and concrete mixes were made with and without air-entraining agents and the air contents determined. While size grading of the sand had a very large influence on air entrainment in a mixture of sand and water alone, this effect was smaller in mortars and very small in concrete mixes. It was found that maximum air was entrained by the 28–48 mesh size sand rather than the 48–100 mesh size.

USE OF PREPACKED AGGREGATE CONCRETE IN MAJOR DAM CONSTRUCTION.....45-14

Price 35 cents.

PAUL BAUMANN—Nov. 1948, pp. 229-236 (V. 45)

This paper sets forth the method of construction of a major gravity dam whereby the average temperature of the mass concrete may be controlled within the mean annual and the dry shrinkage reduced to a fraction of that of conventional concrete by means of prepacking of the coarse aggregate to a density corresponding to 30 percent voids; precooling the aggregate; the intrusion of the coarse aggregate by mortar with a cement content of two sacks per cu yd or less; and the internal post-cooling, curing and draining of the mass concrete. (This process is known as the Prepakt method of construction.)

RATIONAL ANALYSIS AND DESIGN OF TWO-WAY CONCRETE SLABS.....45-15

Price 60 cents.

C. P. SIESS and N. M. NEWMARK—Dec. 1948, pp. 272-316 (V. 45)

A new method for the design of two-way building slabs is proposed and its step-by-step development is described. A new moment distribution procedure is used to compute moments in a number of rectangular slabs continuous over rigid beams. Several variables are studied in the analyses and include: the ratio of sides, the effect of discontinuous edges, the torsional stiffness of the beam, various types of loading, and combinations of panels of various sizes and shapes. Certain conclusions are stated regarding the types of loading to be considered and the values of beam torsional stiffness to be assumed in the development of the design procedure.

Moments obtained in the foregoing analyses are modified to take into account the effect of additional variables. First, the slab moments are increased by various amounts as a result of the deflection of the beams. Next, separate coefficients for dead and live load are replaced by combined coefficients for a live load-dead load ratio of 3.0. Then follows a study to determine the effect of discontinuous edges, and this variable is eliminated from the procedure. Finally all slab moments are reduced by 20 percent in recognition of the redistribution of moments at high load. Additional studies are then made of the distribution of moments across the width of the slab and of the moments to be used for the design of the beams to complete the development of the design procedure.

A proposed design procedure is presented in specification form and is compared with other procedures of a similar nature and with the moment distribution procedure described in the appendix.

CORROSION RESISTANCE TESTS OF CONCRETE FLOORS—WITH AND WITHOUT METALLIC AGGREGATE.....45-16

Price 35 cents.

W. SCRIPTURE, JR. and C. H. SAKRYD—Dec. 1948 pp. 317-324 (V. 45)

An attempt has been made to determine the conditions under which concrete floors can be expected to give good service. Resistance to abrasion was determined on specimens with and without metallic aggregate after exposure to typical reagents selected from the groups described by the Joint Committee as having corrosive effects of an intermediate nature between no attack and severe attack. On the basis of the experimental results the reagents in the Joint Committee report have been classified in relation to anticipated service of concrete floors.

FLEXICORE PRECAST FLOOR AND ROOF SLABS.....45-17

Price 35 cents.

GAYLE B. PRICE—Dec. 1948, pp. 325-340 (V. 45).

This paper describes briefly the design, manufacture, erection procedures and use of Flexicore slabs in precast concrete floor and roof systems. The Flexicore slab has a standard cross section 12 in. wide and 6 in. thick, lightened by two holes 4⅝ in. in diameter running the full length of the slab which is made in inch variations of length up to a maximum of 22 ft 6 in. Reinforcement consists of two bars in the top of the slab for "handling" stresses and two or three prestressed tension bars at the bottom to take care of bending moment.

PROPOSED RECOMMENDED PRACTICE FOR THE APPLICATION OF PORTLAND CEMENT PAINT TO CONCRETE SURFACES.....45-18

Price 35 cents.

REPORT of COMMITTEE 616—Jan. 1949, pp. 353-368, (V. 45)

This report is a studied revision by a reorganized committee of the previous report of Committee 616 published in June 1942, based upon criticisms of that report plus information from more recent tests and experiences. Included are recommendations for appropriate usage, age of concrete, preparation of surface, and the preparation. application and curing of portland cement paint. Three appendixes discuss composition, manufacture and storage and, general characteristics and factors affecting durability.

DRY MORTAR AS A BEARING AND GROUTING MATERIAL.....45-19

Price 35 cents.

BOYD S. BROOKS—Jan. 1949, pp. 369-380, (V. 45)

In 1939, at the navy's new David Taylor Model Basin at Carderock, Md., there was an urgent need for a non-shrinking bearing material to be used under the cast-steel "chairs" which support the rails on which the towing carriages run. Requests for information concerning materials and installation procedures produced such varied answers that investigation was started to find the best material and the best procedures for placing it.

The bearing material selected for use was 1:3 portland cement mortar, with 4.13 gallons of water per sack of cement. This was rammed firmly into place.

This bearing material has now been in place over a period of about 9 years under extremely severe conditions and without exception has proved to be excellent in function and condition.

DIRECT DIMENSIONING OF RECTANGULAR SECTIONS.....45-20

Price 35 cents.

MICHEL BAKHOUM—Jan. 1949, pp. 381-396 (V. 45)

On the basis of the standard theory for reinforced concrete design, methods have been derived for the direct dimensioning of rectangular sections, with or without compression reinforcement, when subjected to simple bending, eccentric compression or eccentric tension. Two cases are considered: when the depth of the section is fixed and when it is not fixed. The conditions leading to minimum reinforcement are also studied. The solutions developed have been simplified by the use of curves which are applicable for pure bending as well as for eccentric compressive or tensile forces and cover any values of f_c, f_s and n. The curves allow making a variety of designs in a relatively short time and thus help in choosing the most economical or most suitable section.

STRENGTHENING BRIDGE SLABS WITH GROUTED REINFORCEMENT.45-21

Price 35 cents.

S. O. ASPLUND—Jan. 1949, pp. 397-408

On the project described the negative reinforcing bars settled as much as 2½ in. out of correct position. Various means for incorporating negative reinforcement at correct height in the finished structure are discussed. The method selected of grouting additional bars in grooves cut with the aid of diamond saw is described. Tests on beam specimens made with some bars encased in the concrete and some bars grouted into diamond-sawed grooves, give results identical with those to be expected for all bars normally encased; ultimate strengths conform closely to the plastic bending capacity of reinforced concrete according to design methods originated by C. S. Whitney.

COMPARATIVE DESIGNS OF A SEGMENTAL SKEWED FRAME CONCRETE BRIDGE BY THE STRAIGHT LINE AND PLASTIC THEORY METHODS.....45-22

Price 35 cents.

MILTON BRUMER—Jan. 1949, pp. 409-420 (V. 45)

The designs of a segmental skewed frame concrete bridge are discussed and compared as developed by both the straight line and plastic theory methods. The plastic theory will lead to rigid concrete frames of more economical proportions capable of sustaining equal or greater loads than frames designed by the straight line method. Effects of volume changes and yielding supports are also of smaller consequence. Deflections may be expected to be greater. The author concludes that further substantiation of the plastic theory is desirable.

SOME FACTORS AFFECTING AIR ENTRAINMENT.....45-23

Price 35 cents.

E. W. SCRIPTURE, Jr. and F. J. LITWINOWICZ—Feb. 1949, pp. 433-444 (V. 45).

A program was undertaken to investigate the influence of various factors on the amount of air entrained in concrete mixes with and without air-entraining agent added. The variables so far studied are slump, cement factor, and sand-total aggregate ratio. The results indicate that the two latter factors are of major importance but that slump has less effect. In general, entrained air content increases with increasing slump, decreases with increasing cement factor, sand-total aggregate ratio.

EXPERIMENTAL AIDS IN STRUCTURAL CONCRETE DESIGN...45-24

Price 35 cents.

R. E. GLOVER, O. J. OLSEN and CARL ZANGAR—Feb. 1949, pp. 445-468 (V. 45).

Statically determinate and indeterminate structures are generally designed by analytical methods. This paper describes several experimental aids which have been used in the design and stress analysis of statically indeterminate structures at the laboratories of the Bureau of Reclamation. These methods are used where analytical procedures cannot conveniently be applied. Described herein are photoelastic methods using both the polariscope and interferometer, the Beggs Deformeter, Stresscoat and electric SR-4 strain gages. Applications are given for each method.

PRACTICES, EXPERIENCES, AND TESTS WITH AIR-ENTRAINING AGENTS IN MAKING DURABLE CONCRETE.....45-25

Price 35 cents.

R. F. BLANKS and W. A. CORDON—Feb. 1949, pp. 469-488 (V. 45)

The authors discuss the advantages of air entrainment with regard to durability, permeability, workability, reduction in alkali expansion, time saving, reduction in water and cement, temperature rise, strength, abrasion resistance, monolithic lightweight concrete and mass concrete. U. S. Bureau of Reclamation experiences are discussed with regard to pumping, transportation and placing, grading of aggregates, factors affecting the amount of air entrained Air-entraining agents and air-entraining cements are discussed briefly. There is a brief section on the measurements of entrained air in concrete with a recommended procedure.

CONCRETE FLOOR FINISHING.....45-26

Price 35 cents.

GERALD MILSOM—Feb. 1949, pp. 489-492 (V. 45).

A concise discussion of the requirements for durable concrete floors including careful selection and control of materials and attention to weather conditions. The importance of proper equipment, working conditions and curing of the finished concrete are emphasized.

PLASTIC FLOW OF CONCRETE AT HIGH OVERLOAD.....45-27

Price 35 cents.

J. R. SHANK—Feb. 1949, pp. 493-500 (V. 45).

New data on the plastic flow of concrete prompted a reconsideration of the assumption that plastic flow is directly proportional to the unit stress applied. Tests are described in which deformations of 6 x 12-in. concrete cylinders were measured while under sustained loads approaching the ultimate strength. Plotted data are shown to demonstrate that the change in the rate of plastic flow appears to be uniform up to three-quarters or more of the ultimate strength with an abrupt change at the "true" ultimate strength. Included are other data indicating that the plastic flow curve has the same general form independent of time and applied load leading to a new consideration in expressing factor of safety and developing beam theories on a factor of safety basis.

CONTROL OF CONCRETE PAVEMENT SCALING CAUSED BY CHLORIDE SALTS.....45-28

Price 35 cents.

B. D. TALLAMY—Mar. 1949, pp. 513-520 (V. 45)

Under modern traffic requirements on heavily traveled roads salt-treated abrasives will not remove ice quickly enough. As maintenance forces have struggled to meet the demand for uninterrupted service the use of straight salts has become increasingly common. Direct applications of up to 600 lb of salt per two-lane mile are required to combat extreme icing conditions. Pavements constructed to withstand the weak brines deposited by salt-treated abrasives cannot stand up under straight salt. New pavements appear more vulnerable than those two or more years old.

The observed resistance to salt action of the oil soaked center streak of uphill traffic lanes led New York state highway engineers to investigate the feasibility of the use of dilute oil applications as a protective measure. Laboratory tests indicate successful results may be expected. In the late summer of 1948 oil spray equipment was developed in time to treat nearly 60 miles of new concrete highways, which should provide a wide base for field observation of the effectveness of the method.

EXPERIENCE WITH AIR-ENTRAINING CONCRETE IN NEW JERSEY...45-29

Price 35 cents.

CHARLES M. NOBLE—Mar. 1949, pp. 521-528 (V. 45)

This paper outlines experience in New Jersey with air-entraining concrete utilized primarily to combat attack by de-icing chemicals. Damage to concrete pavements in New Jersey usually is associated with heavy loads. Freedom from disintegration troubles, except in isolated cases, is attributed to excellent aggregates, rigid laboratory control, mix design, inspection of material and plant equipment and construction supervision at the job site.

Air-entraining portland cement concrete was first specified for a concrete pavement contract in 1945 and since then has been used on many contracts with notable success to prevent deterioration due to scaling. The same high standards for materials, mixture design, laboratory control, field inspection and attention to details must be maintained as with ordinary concrete. Pavements thus far constructed in New Jersey with air entrainment have not

scaled or shown any signs of disintegration when ice control chemicals have been properly mixed with abrasives at a rate not exceeding 75 lb per cu yd of abrasive. It is too early to form judgment but indications are the results achieved fully justify the use of air-entraining concrete.

DESIGN DETAILS FOR ARCHITECTURAL CONCRETE.....45-30

Price 35 cents.

J. J. HOGAN—Mar. 1949, pp. 529-540 (V. 45)

"To achieve beauty with concrete, as with any other material, requires studied design and skillful craftsmanship" are the words of architect Robert D. Murray. This paper directs attention to the matter of "studied design". The skilled craftsmanship required to construct architectural concrete must be preceded by proper design by the architect and engineer. The designer must understand and appreciate the fundamental requirements of this type of construction. This paper discusses considerations appropriate to achieving the desired texture of surface as affected by form materials selected. Ornamentation should be considered in every detail from the standpoint of practicability. Construction joints are inevitable and should be specified in logical locations. Similarly, control joints should be specified with an understanding of where the most likely cracks due to volume change will occur and the design of reinforcement should be coordinated with the location of the control joints. Numerous illustrations support the text.

CONSTRUCTION PRACTICES FOR ARCHITECTURAL CONCRETE.....45-31

Price 35 cents.

E. B. OBERLY—Mar. 1949, pp. 541-552 (V. 45)

Detailed plans and carefully drawn specifications executed by a contractor and superintendent with the know-how of form construction are necessary for good architectural concrete. This paper describes some of the more important construction practices to be considered to achieve the results desired. Form details and concrete quality are discussed in considerable detail, the former being supported by illustrations.

CONCRETE OF THE FUTURE 45-32

Price 35 cents.

ROBERT F. BLANKS—April 1949, pp. 565-568, (V. 45)

Mr. Blanks in his remarks as retiring president, depicts what concrete might be 50 years in the future and discusses the role of the engineer in that era.

CONTROL OF QUALITY OF READY MIXED CONCRETE 45-33

Price 35 cents.

STANTON WALKER—Apr. 1949, pp. 569-580 (V. 45)

Control of quality of ready-mixed concrete is discussed. The objective of the paper is to pose the more important problems of concrete control, especially those peculiar to ready-mixed concrete, with the view of eliciting discussion, rather than to attempt to outline recommended practices. Both central-mixing and transit-mixing are discussed. Attention is directed to the mutuality of responsibility of the producers and users in bringing about effective control.

TESTS OF LIGHTWEIGHT-AGGREGATE CONCRETE DESIGNED FOR MONOLITHIC CONSTRUCTION....45-34

Price 35 cents.

WALTER H. PRICE and WILLIAM A. CORDON—Apr. 1949, pp. 581-600 (V. 45)

The characteristics and origin of lightweight aggregates are described and the comparative results of laboratory tests of strength, insulating value, shrinkage, and weathering resistance of concrete made with light weight aggregates from 17 producers are summarized. Data are included on expanded shale, expanded slag, scoria, pumice, perlite, exfoliated vermiculite and diatoamceous earth.

WHAT HAVE WE LEARNED ABOUT AIR-ENTRAINING CONCRETE?.....45-35

Price 35 cents.

J. F. BARBEE—Apr. 1949, pp. 601-612 (V. 45)

A review of ACI published data on air-entraining concrete, this paper summarizes methods of control and the effect of air entrainment on durability, plasticity and workability, strength, resistance to abrasion and other properties of concrete. The effects of variations in cement and water content, sand-coarse aggregate ratio, mixing time and the use of calcium chloride as an admixture are described. Changes in technique necessary to successfully place, vibrate and finish air-entraining concrete are pointed out. A detailed bibliography follows.

DEVELOPMENTS IN THE MANUFACTURE AND TECHNOLOGY OF CONCRETE MASONRY UNITS.....45-36

Price 35 cents.

JAY C. EHLE—Apr. 1949, pp. 613-620 (V. 45)

The author touches the high spots of developments in the manufacture of concrete masonry units listing the many problems which now face the manufacturer. Rapid growth of the industry in the past ten years and the change from tamping to vibration methods is described. New plants are notable for labor saving equipment compared with those of the middle 1930's. In addition to block manufacture and several methods of curing, in use and proposed, the author mentions briefly: block-handling equipment, maintenance and repair of the plant, and the greater supervisory skill now required for successful operation.

LIGHTWEIGHT-AGGREGATE CONCRETE.....45-37

Price 35 cents.

RALPH W. KLUGE, MORRIS M. SPARKS, and EDWARD C. TUMA—May 1949, pp. 625-644, (V. 45)

Results of studies on lightweight-aggregate concretes made by the National Bureau of Standards at the request of the Housing and Home Finance Agency (formerly the National Housing Agency) are reported in this paper. Aggregates included were expanded clay, shale, and slate; three types of expanded blast-furnace slag; expanded vermiculite and perlite, sintered diatomite and fly ash; and a pumice. Concrete of various proportions were made from these aggregates and determinations were made of the weight per cubic foot, compressive and transverse strength, shrinkage, elastic modulus, absorption, resistance to freezing and thawing and thermal conductivity. Test results are briefly discussed and summarized.

ANALYSIS OF BUILDING FRAMES WITH UNSYMMETRICAL DIFFERENTIAL SETTLEMENT OF THE FOUNDATIONS....45-38

Price 35 cents.

G. A. Leonards—May 1949, pp. 645-652, (V. 45)

The rational design of building frames founded on soil requires a consideration of the effect of settlements caused by the building loads. This paper discusses a procedure for estimating the stresses in such frames due to unsymmetrical differential settlement of the foundations. The procedure is based on the slope-deflection method for analyzing rigid frames, but adapts it to permit a relatively rapid solution by the use of successive approximations.

Continued on p. 26

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Since 1905 that primary objective has been achieved by the combined membership effort. Individually and through committees, and with the cooperation of many public and private agencies, members have correlated the results of research, from both field and laboratory, and of practices in design, construction and manufacture.

The work of the Institute has become available to the engineering profession in annual volumes of ACI Proceedings since 1905. Beginning 1929 the Proceedings have first appeared periodically in the Journal of the American Concrete Institute and in many separate publications.

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Reported by Committee 322 as information and for discussion only. 4 pages, 35 cents per copy. (Reprint from ACI JOURNAL, Nov. 1942)

Proposed Recommended Practice for the Application of Portland Cement Paint to Concrete Surfaces

Adopted as a standard by the 45th annual ACI convention February 24, 1949 subject to ratification of membership by letter ballot to be canvassed about July 1, 1949. 16 pages, 35 cents per copy (Reprint from ACI JOURNAL, Jan. 1949)

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SYNOPSES

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EFFECTS OF MIXING TIME, SIZE OF BATCH AND BRAND OF CEMENT ON AIR ENTRAINMENT ...45-39

Price 35 cents.

E. W. SCRIPTURE, Jr., and F. J. LITWINOWICZ—May 1949, pp. 653-664, (V. 45)

A program, partially described in previous papers, to investigate various factors affecting the amount of air entrained in concrete has been continued. The factors covered in this paper are mixing time, size of batch, and brand of cement. The entrained air rises to a maximum in the early stages of mixing and thereafter decreases. Provided mixing is adequate, the size of batch does not appear to affect the amount of air entrained. Wide variations are found in the amounts of air entrained by different cements and it does not appear possible to correlate these variations with any easily and quickly determinable characteristic of the cement.

A METHOD FOR DETERMINING THE AIR CONTENT OF FRESH AND HARDENED CONCRETE 45-40

Price 35 cents.

ROBERT P. VELLINES and THOMAS ASON—May 1949, pp. 665-672, (V. 45)

A method and equipment are described for determining he air content of fresh and hardened concrete based on the principle that air entrained in concrete reacts to external pressures and that volume changes follow the gas laws. The air entrained in hardened concrete reacts to an external pressure in the same manner as an equal volume of air would if it were not surrounded by hardened cement paste. The volume of the air in the specimen is proportional to the time required for a quantity of gas to escape through an orifice from a pressure of p_1 to a pressure of p_2 at a constant temperature.

STUDY OF DUSTY CONCRETE CEILINGS 45-41

Price 35 cents.

JACOB FELD—May 1949, pp. 673-680, (V. 45)

In a large Eastern housing project of two-story concrete buildings considerable trouble was experienced from the "dusting" of first floor ceilings which were poured against new pressed-board forms. The experimental work to isolate the cause, the results of the tests and their interpretation indicated that the dusting was caused by an acidic resinous substance in the surface of the form board. The substance on the surface of the board appeared to be neutralized when the form was used for the first time, inasmuch as reused form boards gave little trouble. In concluding, the author recommends that new types of form material be checked for possible chemical reaction before use in large amounts.

ACI publications in large current demand

Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-48)

Approved as a Standard of the American Concrete Institute at its 44th Annual Convention February 25, 1948. Ratified by letter ballot June 29, 1948. It is of large format, bound to lie flat and presents typical engineering and placing drawings with discussion calling attention to important considerations in designing practice. It was prepared to simplify, speed, and effect standardization in detailing. It is believed to be the only publication of its kind in English. It is meeting wide acclaim among designers, draftsmen and in engineering schools. Price—$3.00, to ACI Members—$1.75.

ACI Standards—1948

202 pages, 6x9 reprinting current ACI Standards: Building Code Requirements for Reinforced Concrete (ACI 318-47); Minimum Standard Requirements for Precast Concrete Floor Units (ACI 711-46); five recommended practices: Use of Metal Supports for Reinforcement (ACI 319-42); Measuring, Mixing and Placing Concrete (ACI 614-42); Design of Concrete Mixes (ACI 613-44); Construction of Concrete Farm Silos (ACI 714-46); Winter Concreting Methods (ACI 604-48); and two specifications: Concrete Pavements and Bases (ACI 617-44) and Cast Stone (ACI 704-44)—nine ACI Standards in one book, $2.00 per copy—to ACI Members, $1.25. The "Detailing Manual" (ACI 315-48) is available separately only at its established price—see above.

Air Entrainment in Concrete (1944)

92 pages of reports of laboratory data and field experience including a 31-page paper by H. F. Gonnerman, "Tests of Concretes Containing Air-entraining Portland Cements or Air-entraining Materials Added to Batch at Mixer," and 61 pages of the contributions of 15 participants in a 1944 ACI Convention Symposium, "Concretes Containing Air-entraining Agents," reprinted (in special covers) from the ACI JOURNAL for June, 1944, $1.25 per copy, 75 cents to members.

Air Entrainment in Concrete—Book 2 (1947)

A 204-page compilation of information on air entrainment published after Book 1 (1944), including: "Field Use of Cement Containing Vinsol Resin", and "Laboratory Studies of Concrete Containing Air-Entraining Admixtures", by Charles E. Wuerpel, "Entrained Air in Concrete", a foreword and 14 short papers presented at the 1946 Convention; and discussion of the symposium, reprinted from ACI JOURNALS for September 1945, Feb., June and Dec. Part 2, 1946. $2.25 per copy, $1.50 to ACI Members.

ACI Manual of Concrete Inspection (July 1941)

This 140-page book (pocket size) is the work of ACI Committee 611, Inspection of Concrete. It sets up what good practice requires of concrete inspectors and a background of information on the "why" of such good practice. Price $1.00—to ACI members 75 cents.

"The Joint Committee Report" (June 1940)

The Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete submitting "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete," represents the ten-year work of the third Joint Committee, consisting of affiliated committees of the American Concrete Institute, American Institute of Architects, American Railway Engineering Association, American Society of Civil Engineers, American Society for Testing Materials, Portland Cement Association. Published June 15, 1940, 140 pages. Price $1.50—to ACI members $1.00.

Reinforced Concrete Design Handbook (Dec. 1939)

This report of ACI Committee 317 is in increasing demand. From the Committee's Foreword: "One of the important objectives of the committee has been to prepare tables covering as large a range of unit stresses as may be met in general practice. A second and equally important aim has been to reduce the design of members under combined bending and axial load to the same simple form as is used in the solution of common flexural problems."—132 pages, price $2.00—$1.00 to ACI members.

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The complex and sometimes unstable compounds formed when cement hydrates make analysis difficult. The authors studied these and related hydrous solids by thermal analysis with results indicating that there is a hitherto undiscovered phase characterized by an endothermic bulge at 130-140 C.

An Investigation of Hydrating Cements and Related Hydrous Solids by Differential Thermal Analysis*

By GEORGE L. KALOUSEK†, CURTIS W. DAVIS, Jr.‡ and WILLIAM E. SCHMERTZ‡

SYNOPSIS

Differential thermal analysis was used to identify some of the hydrous solids formed in cements and to follow partially the course of some of the hydration reactions. Pastes of nine commercial portland cement clinkers with and without additions of gypsum were tested at 1, 3, 7, 28 and 90 days and 9 months. Among the individual solids prepared and studied were the calcium sulfoaluminates, hydrous calcium aluminates, lime-silica gels made from different starting materials, with and without additions of Al_2O_3, SO_3 and Na_2O. Thermal analysis results show that the isometric tricalcium aluminate hexahydrate does not form in hydrating cements. The trisulfate form of the calcium sulfoaluminate and the solid solution, $3CaO \cdot Al_2O_3 \cdot CaSO_4 \cdot 13H_2O$-$3CaO \cdot Al_2O_3 \cdot Ca(OH)_2 \cdot 12H_2O$, appear as metastable products only. The stable sulfate-bearing phase has not yet been isolated but presumably is characterized in thermal analysis by an endothermic bulge at 130-140 C. A possible role of Fe_2O_3 in the formation of this new phase is considered. The lime-silica gels may occur in two forms, one exhibiting two distinctive thermal changes and the other not showing any marked changes; the former is the more unstable of the two, transforming in prolonged tests to the latter. Alumina appears to stabilize the less stable form of these gels.

INTRODUCTION

Cement technologists are not always able to account satisfactorily for a varied performance of concrete in service. This may be attributed largely to the dearth of fundamental data on the chemical and physical properties of some of the hydrous reaction products comprising concrete. It has been recognized for some time that means other than microscopical

*Received by the Institute Feb. 5, 1949. Title No. 45-42 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 10, June 1949, *Proceedings* V. 45, Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Sept. 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

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or those based on X-ray diffraction would be required for the desired characterization or identification of the potential phases not yet fully identified. In recent years differential thermal analysis has been applied successfully in studies of clay minerals and other hydrates and solids. It seemed that this method should prove equally valuable for identifying some of the products of reaction of cement and water and was accordingly applied in the present investigation. This report describes the application of thermal analysis to a study of hydrating cements and gives the results obtained.

An understanding of the nature of the hydrous lime-silica products in cements is of utmost importance to cement technologists. Steinour's recent comprehensive review[1] of reports on the system lime-silica-water shows that the data on these products are contradictory in part and inconclusive. Application of such results, notwithstanding their limitations, for interpreting the behavior of concrete is further complicated by the still largely unknown effects of the constituents (as Al_2O_3, Fe_2O_3, SO_3, Na_2O and K_2O) extraneous to the ternary system but present in cements. Forsen[2] studied the effects of some of these constituents and concluded, among other summarizations, that alumina and silica co-precipitate in cements under certain conditions. Kalousek[3] has shown that Na_2O precipitates partly from solution in the system Na_2O-CaO-SiO_2-H_2O as a quaternary gel-like phase and has a marked influence on the composition of the gel even when present in solution in relatively low concentration. The tests in part of the present investigation were designed to study by thermal analysis the effects of Na_2O, SO_3 and Al_2O_3 on the hydrous lime-silica products. Also the influence of the nature of the starting forms of silica was considered.

TABLE 1—OXIDE AND CALCULATED COMPOUND COMPOSITIONS OF THE CEMENT CLINKERS

No.	SiO_2	Fe_2O_3	Al_2O	CaO	MgO	Na_2O	K_2O	SO_3	Loss on ignition	C_3S*	C_2S	C_3A	C_4AF
1	21.7	2.5	6.6	64.4	3.1	0.21	0.21	0.93	0.3	56	19	13	8
3	22.2	2.5	6.2	63.6	3.3	0.52	0.20	0.18	0.6	49	27	12	8
4	23.5	2.2	5.6	66.1	1.4	0.14	0.15	0.05	0.7	50	27	11	7
5	21.3	3.8	6.4	64.2	1.3	0.19	0.70	0.65	1.1	51	23	11	12
8	21.1	3.2	6.0	66.5	0.9	0.26	0.40	0.22	1.0	65	12	10	10
9†	23.3	4.4	4.6	65.0	1.4	0.05	0.18	0.03	1.1	48	31	5	13
10†	28.0	1.9	2.1	65.6	1.8	0.05	0.23	0.17	0.3	37	52	2	6
11†	24.0	4.6	5.3	62.1	1.6	1.01	0.10	0.67	0.5	46	26	6	15
12	23.7	0.4	4.9	65.5	2.9	0.07	0.11	0.08	2.2	54	28	12	1

*The abbreviated form of compound formula is used with $C = CaO$, $S = SiO_2$, $A = Al_2O_3$ and $F = Fe_2O_3$.
†Clinkers No. 9, 10 and 11 were supplied by the Portland Cement Assn. and are No. 15623, 15670 and 16839, respectively, in their series. The analyses of these 3 clinkers given here are those reported previously by Lerch.[7]

The role of gypsum in the chemistry of cements has been studied extensively. However, the exact nature of the hydrous sulfate-bearing products in cements has remained in doubt. In this connection the reports by Jones[4] and Kalousek[5] on the relative stabilities of the calcium sulfoaluminates and the related hydrous calcium aluminates, particularly the isometric tricalcium aluminate hexahydrate, are of interest. It seemed appropriate, therefore, to apply the present method of study to an investigation of the potential hydrous alumina- and sulfate-bearing solids in cements.

The investigation consisted essentially of synthesizing the phases mentioned and analyzing them thermally. These results in turn were compared with those obtained on cement clinker pastes hardened for different periods with and without addition of gypsum.

MATERIALS

Cement clinkers

Nine commercial portland cement clinkers were selected for this study. The oxide and potential compound compositions computed by the method of Bogue[6] are given in Table 1. The clinkers were ground to specific surface areas of 2800-3300 sq cm per g; measurements were made with a Blaine fineness meter calibrated with National Bureau of Standards cement sample No. 114e following Federal Specifications for Cements SS-C-158b.

A portion of each clinker was intimately mixed with an addition of gypsum of analytical purity. All clinkers were treated with 1.8 percent SO_3 as gypsum, some with 2.2, 3.5, 5.0 or 15.0 percent as required for specific tests.

Starting materials for individual hydrous solids

The starting materials for the individual hydrous solids were $Ca(OH)_2$, $NaOH$, $NaAlO_2$, $Al_2O_3 \cdot xH_2O$, $Fe_2O_3 \cdot xH_2O$, $4CaO \cdot Al_2O_3 \cdot Fe_2O_3$, $2CaO \cdot Fe_2O_3$, solid silicic acid and freshly boiled distilled water. The solids met or exceeded U.S.P. purity standards or were made of such reagents. Calcium carbonate low in alkalies was ignited and the resulting CaO served for making solutions of $Ca(OH)_2$. The $NaOH$ was prepared as a clear saturated solution and diluted as required with distilled water. The $4CaO \cdot Al_2O_3 \cdot Fe_2O_3$ and $2CaO \cdot Fe_2O_3$ were prepared according to standard methods.[8] An alumina-silica gel consisting of 8.1 percent Al_2O_3, 88.4 percent SiO_2 and 3.7 percent loss on ignition, and a silica gel consisting of 89.8 percent SiO_2, 0.40 percent non-volatile with HF and 10.0 percent loss on ignition were also used.

PROCEDURE OF STUDY

Processing cement clinkers

The cement clinkers with and without addition of 1.8 percent SO_3

as gypsum were mixed for 3 minutes with water, using a water-cement ratio of 0.50 by weight. The resulting slurry, weighing about 125 g, was divided into 8 glass vials which were then tightly stoppered and tested at ages of 1, 3, 7, 28 and 90 days and 9 months. The sample was carefully separated from the glass of the vial and finely crushed in a mortar. The powdered material was washed in a sintered glass filter 4 times with acetone, then 2 times with ethyl ether and finally conditioned by placing it in a gas-tight receptacle at room temperature and pumping off continuously for 1 hour the residual gases at a pressure of about 0.01 mm, or less. In special tests, deviations were made from the foregoing procedure and will be discussed later.

The individual solids were processed similarly except that absolute ethyl alcohol was substituted for acetone for washing the preparations prepared in the presence of $NaOH$. Chemical analysis for Na_2O showed that some Na_2O-bearing phase was apparently precipitated by the acetone from the residual aqueous phases in the gelatinous solids after filtration, but not by the ethyl alcohol.

Preparation of individual hydrous solids

The isometric tricalcium aluminate hexahydrate, $3CaO \cdot Al_2O_3 \cdot 6H_2O$, was prepared by autoclaving at 150 C for 3 days an intimate mixture of CaO and $Al_2O_3 \cdot xH_2O$ (aluminum hydroxide) in a molar ratio of $3CaO$: $1Al_2O_3$ in the presence of excess water. Microscopic examination of the product failed to reveal any impurity. The loss on ignition corresponded to a molar ratio of 6.04 H_2O:1Al_2O_3.

Members of the sulfoaluminate solid solution, $3CaO \cdot Al_2O_3 \cdot Ca(x)SO_4 \cdot (1\text{-}x)\ (\overline{OH})_2 \cdot 13\text{-}12H_2O$*, were prepared according to a method described previously[5]†. The SO_3 as gypsum was dissolved in a $NaOH$ solution supersaturated with $Ca(OH)_2$ and containing 0.04 to 0.5 equivalent of OH^- per liter as $NaOH$ and $Ca(OH)_2$. Alumina as $NaAlO_2$ in a SO_3:Al_2O_3 ratio of 1.0 was added as the last constituent to the mixture.

The trisulfate form of the sulfoaluminate, $3CaO \cdot Al_2O_3 \cdot 3CaSO_4 \cdot 31H_2O$, was prepared in prism crystals of moderately large size (roughly 3 by 200 microns) from a mixture of solutions of gypsum, $Ca(OH)_2$ and $NaAlO_2$. The SO_3:Al_2O_3 molar ratio of the mix was maintained at a value of 3.00 and the concentration of $Ca(OH)_2$ was so fixed that about 0.5 g of CaO per liter was present in the equilibrium solution.

This study was concerned with the hydrous lime-silica products (the cement gels) formed in the presence of a saturated $Ca(OH)_2$ solution

*In this formula x has any positive value of 1 or less. Two OH^- replace one SO_4^{--}, or vice versa, in the lattice. When $x=1$ the member so represented is the monosulfate, $3CaO \cdot Al_2O_3 \cdot CaSO_4 \cdot 13H_2O$. In the absence of SO_4^{--} ($x=0$) the tetracalcium aluminate hydrate, $3CaO \cdot Al_2O_3 \cdot Ca(OH)_2 \cdot 12H_2O$, is represented by the formula.

†The amounts of the oxide components for different members of the solid solution are given in Table 2 (to be discussed under results). The SO_3 is supplied by gypsum and the CaO represents the total as $Ca(OH)_2$ and gypsum.

only, a condition which is known to prevail in hydrating cements. To assure a saturated solution it was necessary to have crystalline $Ca(OH)_2$ present as a second solid phase. The published data[1] reveal that the CaO:SiO_2 molar ratio of the hydrous lime-silica gels may be as high as 2.0 or even slightly higher. The lime was added so that the over-all CaO:SiO_2 ratio was between 2.2 and 2.5, allowance being made for the $Ca(OH)_2$ required for saturating the aqueous phase. The silica, in an amount of 0.5 g as SiO_2, was added to the suspension of $Ca(OH)_2$ in saturated $Ca(OH)_2$ solution and the mixtures were shaken once or twice daily. The mixtures were filtered at 7, 14, 28 and 60 days, and some at 6 months. No attempt was made to prepare the lime-silica gels free of $Ca(OH)_2$ or to determine the amount of free silica.

In a second method of preparing these solids, the silica gel was replaced by a solution of silicic acid in dilute $Ca(OH)_2$ made according to the procedure described by Flint and Wells.[9] The same overall CaO:SiO_2 ratio was maintained for the solid phases.

Further tests were made to ascertain the effect of Al_2O_3 on the lime-silica gel by substituting the alumina-silica gel for the silica gel, the conditions otherwise remaining the same.

Gypsum and $NaOH$, singly or together, were added to the lime-silica mixtures to ascertain the effects of these two constituents on the reaction products. The amounts added were varied and will be given with the results.

The $4CaO \cdot Al_2O_3 \cdot Fe_2O_3$ and $2CaO \cdot Fe_2O_3$ were used in several mixtures to ascertain the effect of Fe_2O_3 on one of the potential reaction products of cements. The details are reported with the results. No attempt was made to prepare a pure hydrous compound containing Fe_2O_3.

Chemical analyses

Standard procedures were followed in analyzing the starting materials and some of the pure phases. Analyses of the cement clinkers were made according to A.S.T.M. Methods C 114-46T.

Thermal analysis

Two nichrome wire wound furnaces as described by Berkelhamer[10] were used for thermal analysis. The rate of temperature change of the samples during the test attained a value of about 7 C per minute at 100 C. In the temperature interval of 100 to 300 C the rate increased to 12.5 C per minute and then remained nearly constant at this value for the remainder of the test. The test sample (0.15 to 1.00 g depending on the bulk density) was placed in one compartment of the cell in which rested one junction of a differential chromel-alumel couple, the other junction was in the companion compartment of the cell which contained ignited Al_2O_3 as the reference inert material. The emf developed by this dif-

ferential couple was recorded automatically by a potentiometer. Another potentiometer driven synchronously with that just mentioned recorded continuously the temperature of the sample.* The temperatures at which the thermal changes occurred were obtained from this record.

RESULTS

Potential hydrous solids of cements

Calcium sulfoaluminates and other alumina-bearing phases—Table 2 gives the initial compositions of representative mixtures used to prepare the solid solution of the sulfoaluminate including the end member, $3CaO \cdot Al_2O_3 \cdot Ca(OH)_2 \cdot 12H_2O$, and the compositions of the solids obtained. The highest SO_3 content in the solid solution corresponded to an SO_3:Al_2O_3 molar ratio of 0.81, somewhat low compared to that of the monosulfate. The series as presented, however, are sufficiently complete to interpret the thermal analysis results on cements.

Because change in the OH^- concentration of the mixtures alters the SO_3:Al_2O_3 ratio of the solid solution the question arose whether alteration in the concentration of $NaOH$ in the mixtures for preparing the $3CaO \cdot Al_2O_3 \cdot Ca(OH)_2 \cdot 12H_2O$ might not have an effect on the composition of this solid. Tests were carried out over a OH^- concentration range of 0.04 to 0.5 equivalent per liter but no significant difference was observed in chemical analysis or thermal analysis among these samples. Representative samples of this phase and other members of the solid solution were analyzed for Na_2O but this constituent was absent from all samples washed with ethyl alcohol. It seems unlikely, therefore, that Na_2O is a constituent of the foregoing products.

Thermal analysis curves of representative samples of the preparations just described are given in Fig. 1. The SO_3-rich (0.81 SO_3:Al_2O_3) solid

TABLE 2—COMPOSITIONS OF INITIAL MIXTURES CONTAINING Na_2O, CaO, Al_2O_3 AND SO_3 AND OF THE SOLIDS OBTAINED

Composition of initial mixture				Composition of solid phases in moles			
CaO g/l	Al_2O_3 g/l	SO_3 g/l	OH^- equiv./l	CaO	Al_2O_3	SO_3	H_2O
1.2	0.100	0.080	0.04	3.90	1.00	0.81	11.9
0.50	0.100	0.080	0.20	*	1.00	0.69	....
0.30	0.100	0.080	0.29	3.94	1.00	0.52	11.8
0.27	0.100	0.080	0.41	*	1.00	0.44	....
0.27	0.100	0.000	0.40	4.02	1.00	0.00	13.8
0.90	0.100	0.238	0.04	6.00	1.00	3.00	31.0
3.00†	1.00	....	....	3.00	1.00	0.00	6.04

*Excess $Ca(OH)_2$ present.
†$3CaO \cdot Al_2O_3 \cdot 6H_2O$ was prepared at 150 C.

*Since the emf developed by the differential thermocouple is automatically balanced against the emf of the cell of the potentiometer, it is possible to measure the emf of one junction of this differential couple similarly balanced by a second potentiometer for determining the temperature of the sample.

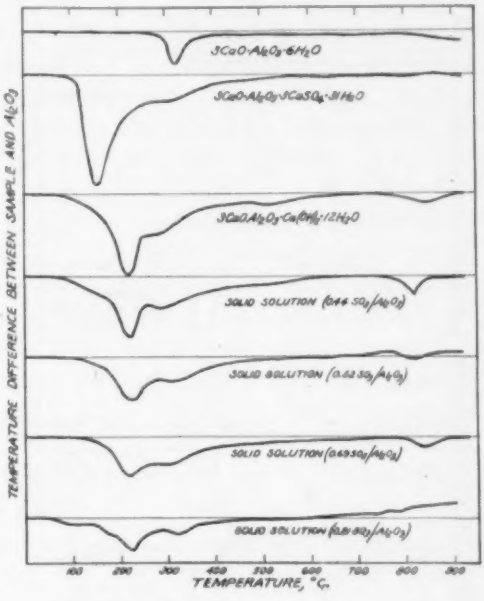

Fig. 1—Thermal analysis curves of members of the calcium sulfoaluminate solid solution of $3CaO \cdot Al_2O_3 \cdot 3CaSO_4 \cdot 31H_2O$ and $3CaO \cdot Al_2O_3 \cdot 6H_2O$

solution shows one dehydration bulge at 220 C and another, but less marked, at about 330 C. The bulge at 210-220 C is present for all members of the series but its size increases as the amount of SO_3 decreases to a value of about 0.4 SO_3:Al_2O_3. The members poorer in SO_3 than 0.4 SO_3:Al_2O_3, including the SO_3-free compound, show no further variation in the extent of this bulge and little, if any, differences in the curve. Such a trend was not observed for the 330 C bulge which decreased in extent and appeared more as an inflection in the curves at increasingly lower temperatures with decreasing SO_3 content.

Fig. 1 also includes a thermal curve for the $3CaO \cdot Al_2O_3 \cdot 3CaSO_4 \cdot 31H_2O$. The dehydration bulge at 160 C serves well to characterize this compound. Its thermal change is considerably more extensive than that for the solid solution. This phase also shows a tendency towards the formation of a bulge at about 270-300 C.

The thermal curve for a mixture of 10 percent $3CaO \cdot Al_2O_3 \cdot 6H_2O$ in ignited Al_2O_3 is included in Fig. 1. The endothermic bulge at 315-330 C characterizing the isometric phase is relatively very large and, for this reason, as little as 0.5 percent of this phase can be detected in cements by means of thermal analysis.

Hydrous lime-silica products—In presenting the results on the hydrous lime-silica gels particular attention is called to the effect of the nature of the starting raw material on the products obtained, especially at the earlier test ages. The lime-rich hydrous gels precipitated from the

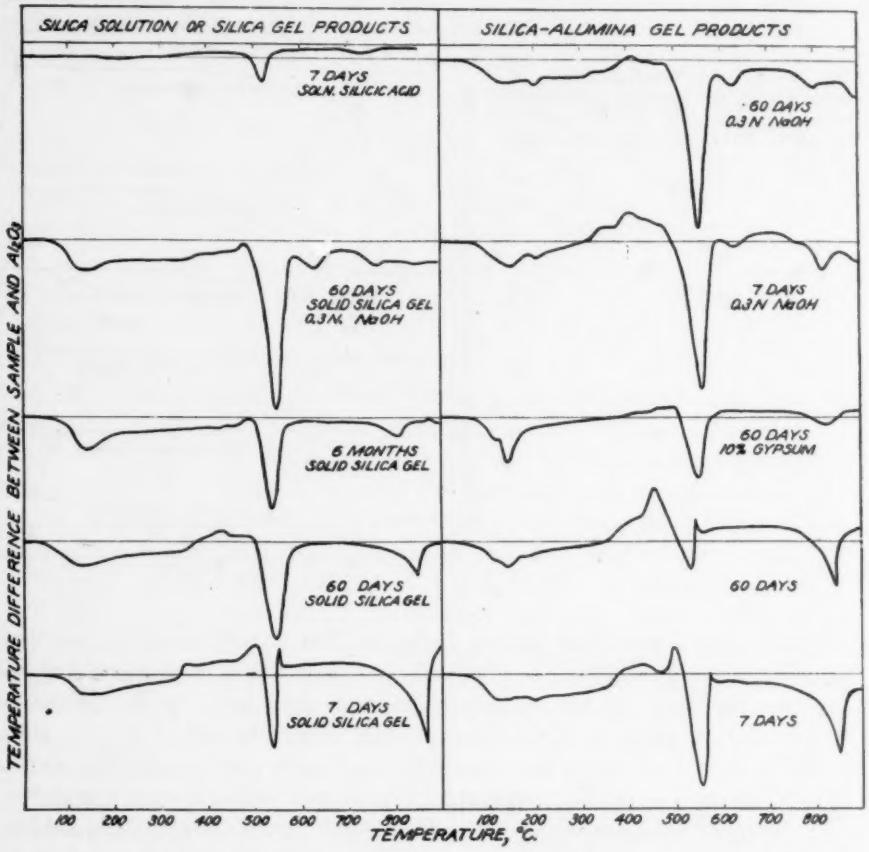
SILICA SOLUTION OR SILICA GEL PRODUCTS
SILICA-ALUMINA GEL PRODUCTS
7 DAYS SOLN. SILICIC ACID
60 DAYS 0.3 N NaOH
60 DAYS SOLID SILICA GEL 0.3 N. NaOH
7 DAYS 0.3 N NaOH
6 MONTHS SOLID SILICA GEL
60 DAYS 10% GYPSUM
60 DAYS SOLID SILICA GEL
60 DAYS
7 DAYS SOLID SILICA GEL
7 DAYS
TEMPERATURE DIFFERENCE BETWEEN SAMPLE AND Al2O3
100 200 300 400 500 600 700 800
100 200 300 400 500 600 700 800
TEMPERATURE, °C.

Fig. 2—Thermal analysis curves of products made from $Ca(OH)_2$ and either a solution of silicic acid, solid silica gel or alumina-silica gel at different ages of test with and without addition of $NaOH$ and gypsum

aqueous solution of silicic acid by $Ca(OH)_2$ gave thermal curves which showed only very slight thermal changes (excepting that for $Ca(OH)_2$ present as a second solid phase). In other words, this gel when heated to 900 C does not undergo any transformation characterized by either adsorption or evolution of a significant amount of heat compared to that of the other solids being considered. A curve for one of these products is included in Fig. 2.

The reaction product obtained from the solid silica gel and $Ca(OH)_2$, on the other hand, shows some striking and reproducible thermal changes. Of particular interest is the prolonged and irregular exothermic bulge starting at temperatures of about 350 to 400 C and extending to about 500 C and the endothermic peak or bulge appearing at temperatures of

800 to 890 C*. Fig. 2 gives curves for the 7- and 60-day and 6-month products. The thermal bulges mentioned decrease in extent as the age of test is increased and are small at 6 months. Addition of gypsum had no significant effect on these curves at ages of 7 and 28 days, the only tests made. Addition of *NaOH* (0.3N solution) altered extensively the thermal changes as shown by the result obtained on the 60-day sample. The endothermic bulge at 630 C appeared only in samples prepared in the presence of 0.3N *NaOH* solution and is considered, therefore, to characterize a Na_2O-bearing phase. This bulge did not appear in any other product. The composition of this solid as deduced from a previously reported[3] relation between composition of solid and solution is about $0.2\ Na_2O{:}1CaO{:}1SiO_2{:}nH_2O$. The 0.3N value was selected because it corresponded to the highest concentration of Na_2O theoretically possible in a hydrating cement containing 1 percent Na_2O as computed on the basis of certain assumptions[3]. Tests were also made using a concentration of 0.15N *NaOH*; the thermal curves for the resulting products were similar to those without addition of *NaOH* except for a barely discernible bulge at 630 C for only a few of the samples prepared in this series. Addition of gypsum did not alter materially the previously described effect of the *NaOH*.

Substitution of the alumina-silica gel (8.1 percent Al_2O_3) for the silica gel gave results differing in two respects from those obtained on the products of lime and silica gel. At ages of 7, 14, and 28 days there was observed an endothermic bulge at about 470 C, and a small endothermic bulge at 200-210 C besides the other thermal changes found for the lime-silica gel products at corresponding ages. The 200-210 C bulge indicates the presence of $3CaO \cdot Al_2O_3 \cdot Ca(OH)_2 \cdot 12H_2O$, but the other thermal change was not accounted for.

A comparison of corresponding curves for the lime-silica gel and lime-alumina-silica gel products shows a marked difference with increasing age of test. The Al_2O_3-containing solid does not show a decrease in the 400-500 C exothermic bulge or the 800 or 890 C endothermic bulge.

The Al_2O_3-bearing gel if prepared in 0.3N *NaOH* gave thermal changes nearly the same as those observed for the companion samples not containing Al_2O_3. In Fig. 2 are included curves for the 7- and 60-day products in 0.3N *NaOH*. The changes between 7 and 60 days are not particularly large, except for a decrease in the size of the 400-500 C bulge. Addition of 10 percent gypsum to the solids in the 0.3N *NaOH* solution resulted in the formation of a product showing a nearly identical thermal behavior to that shown when gypsum was not added. Ten percent addition of gypsum without *NaOH* to the lime- and alumina-silica gel

*This bulge could be caused partly by decomposition of $CaCO_3$. The contribution to the bulge by the ordinary amount of CO_2 contamination (chemical analysis showed 0.3 to 1.2 percent CO_2), however, was found to be comparatively very small in control tests.

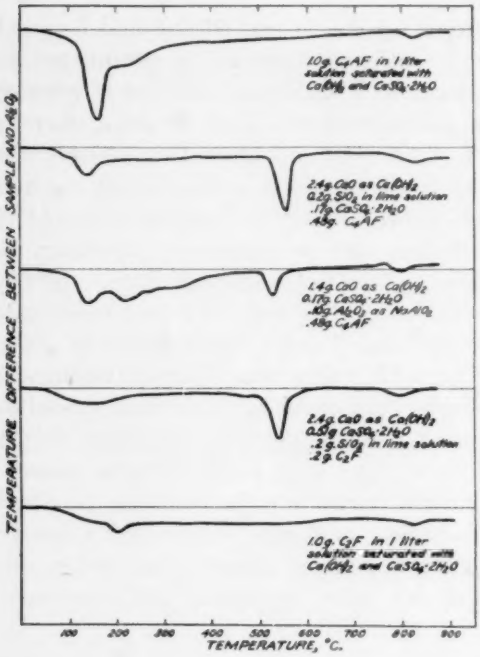
1.0g C4AF in 1 liter solution saturated with Ca(OH)2 and CaSO4·2H2O
2.4g CaO as Ca(OH)2
0.2g SiO2 in lime solution
.17g CaSO4·2H2O
.45g C4AF
2.4g CaO as Ca(OH)2
0.17g CaSO4·2H2O
.10g Al2O3 as NaAlO2
.45g C4AF
2.4g CaO as Ca(OH)2
0.5g CaSO4·2H2O
.2g SiO2 in lime solution
.2g C2F
1.0g C2F in 1 liter solution saturated with Ca(OH)2 and CaSO4·2H2O
TEMPERATURE DIFFERENCE BETWEEN SAMPLE AND Ag
100 200 300 400 500 600 700 800 900
TEMPERATURE, °C.

Fig. 3—Thermal analysis curves of products made from $2CaO \cdot Fe_2O_3$ or $4CaO \cdot Al_2O_3 \cdot Fe_2O_3$ in 7-day tests

gave a product, as shown by the curve in Fig. 2, probably the same as that without Al_2O_3. In this sample the Al_2O_3 apparently was combined as the trisulfate as shown by the endothermic bulge at 145 C.

Calcium hydroxide—It is well known that $Ca(OH)_2$ is present in cement as a hydrolytic product of $3CaO \cdot SiO_2$ and possibly also of $2CaO \cdot SiO_2$ in high-alkali cements. Identification of $Ca(OH)_2$ by thermal analysis is based on the endothermic peak due to dehydration which generally occurs at 560 to 590 C. However, when this compound is present in cements in small amounts and presumably in a finely subdivided state at the early periods of about 1 day, the bulge is observed at a temperature as low as 515 C. (Fig. 7, which shows results to be discussed, serves also to illustrate the "growth" of the dehydration bulge of $Ca(OH)_2$.)

Tests on Fe_2O_3-bearing mixtures—As this investigation advanced it seemed desirable to make some tests in which pure crystalline $2CaO \cdot Fe_2O_3$ and $4CaO \cdot Al_2O_3 \cdot Fe_2O_3$ were used with and without additions of gypsum and also in solutions of silicic acid and $Ca(OH)_2$. Thermal curves on representative products are given in Fig. 3. A 7-day reaction product of 1.0 g $2CaO \cdot Fe_2O_3$ in a solution saturated with respect to $Ca(OH)_2$ and $CaSO_4 \cdot 2H_2O$ gave a thermal curve showing an endothermic bulge at 195 C. A parallel experiment in which $4CaO \cdot Al_2O_3 \cdot Fe_2O_3$ replaced the $2CaO \cdot Fe_2O_3$ gave a product showing only the dehydration bulge of the

trisulfate. However, the $4CaO \cdot Al_2O_3 \cdot Fe_2O_3$ if hydrated in the presence of lower amounts of SO_3 (the SO_3:Al_2O_3 not exceeding 1.0) always yielded a product showing the endothermic bulge at 135-140 C, regardless of the presence of other constituents. Addition of Al_2O_3 as $NaAlO_2$ in such tests gave products showing the endothermic thermal changes at 135-140 and 210-220 C. The $2CaO \cdot Fe_2O_3$ did not show much tendency toward formation of a product giving the 135-140 C bulge. It appears that the two constituents Al_2O_3 and Fe_2O_3, if present together as in $4CaO \cdot Al_2O_3 \cdot Fe_2O_3$, lead to the formation of the phase giving the en-

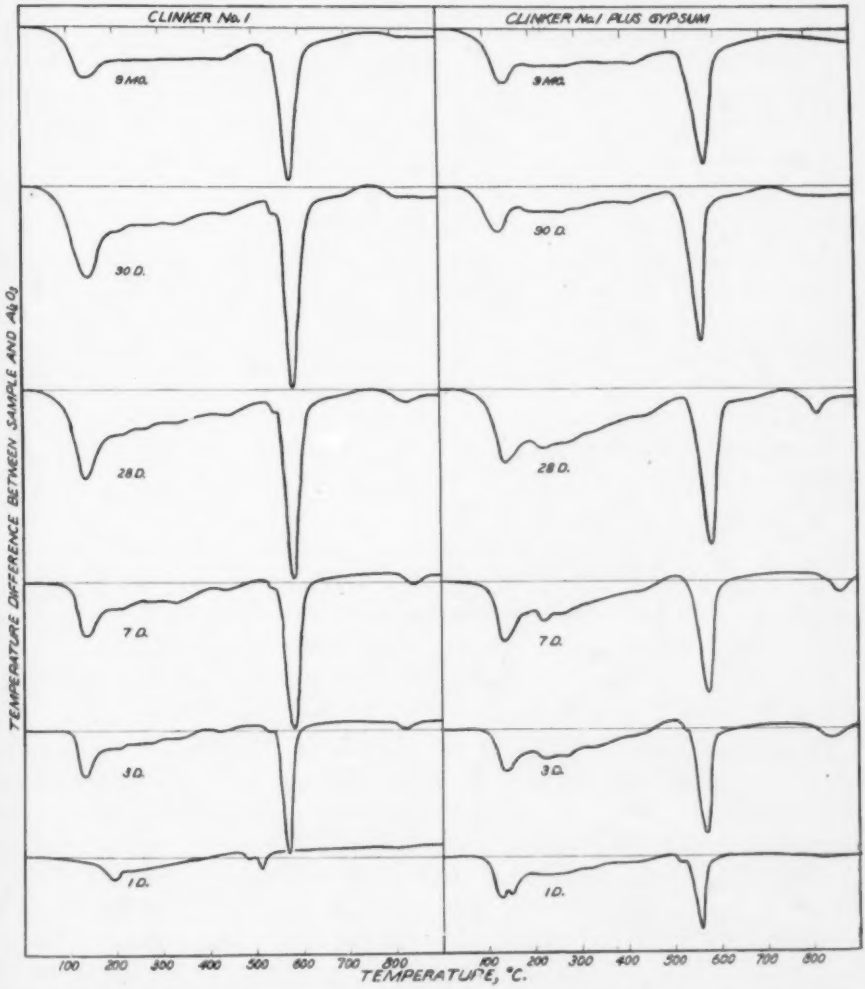

Fig. 4—Thermal analysis curves of cement clinker No. 1 at different ages of test with and without addition of 1.8 percent SO_3 as gypsum

dothermic bulge at 135-140 C. More work remains to be done to ascertain definitely the conditions under which this phase occurs.

Partially hydrated cement clinkers

Fig. 4, 5 and 6 present thermal curves for the clinkers illustrating the more pertinent thermal changes and how these varied in extent as the age of test was increased from 1 day to 9 months. The cement clinkers with and without addition of gypsum are considered separately; the effect of gypsum is shown by comparing the results for the clinkers with and without addition of gypsum.

Cement clinkers without addition of gypsum—The clinker pastes without added gypsum showed an endothermic bulge at about 130-140 C which appeared at 3 days and generally increased in size, attaining a maximum at either 28 or 90 days, but at 9 months there was a moderate to marked decrease in the extent of this bulge. The 130-140 C bulge could be attributed to the evaporation of capillary water present in the lime-silica gel in the cement and actually is due partly to this cause. Further explanation of the cause of this thermal change will be given.

An endothermic bulge at 195-200 C of slight to moderate extent appeared for all clinkers except No. 10 at 1 day, but the thermal change at 130-140 C did not occur at this age of test. Clinkers 11 and 12 showed relatively large endothermic bulges at 210-220 C indicative of the presence of SO_3-poor members of the solid solution. Clinkers 1,3,5 and 8 showed the presence of this phase at 3 to 28 days in moderate amounts and clinkers 4, 9 and 10 failed to give any significant thermal changes at 210-220 C. Fig. 4, 5 and 6 show the variations in the extent of this bulge.

Only clinker 4 at 28 and 90 days and clinker 9 at 90 days and 9 months showed the endothermic bulge at 315-330 C characterizing $3CaO \cdot Al_2O_3 \cdot 6H_2O$ as illustrated by curves for clinker 4 in Fig. 5.

A small endothermic reaction occurring at about 415-430 C generally for 90-day and 9-month samples is attributed to the dehydration of $Mg(OH)_2$ formed from the MgO in the cements.

A small endothermic bulge at 480-515 C shown by clinkers 1, 5 and 8 at 1 through 28 days has not been associated with any phase considered in this study. Nor has any clue been uncovered which might even suggest the identity of the phase giving this bulge.

The endothermic bulge at 515-590 C (making allowance for the one not yet associated with any known potential phase) shows the presence of $Ca(OH)_2$. The clinkers of the high Fe_2O_3 content (No. 9 and 11) showed smaller amounts of $Ca(OH)_2$ at 1 day than the other clinkers. The amount of this compound, however, increased for all clinkers as the age of test was increased to 28 or 90 days. At 9 months (or 90

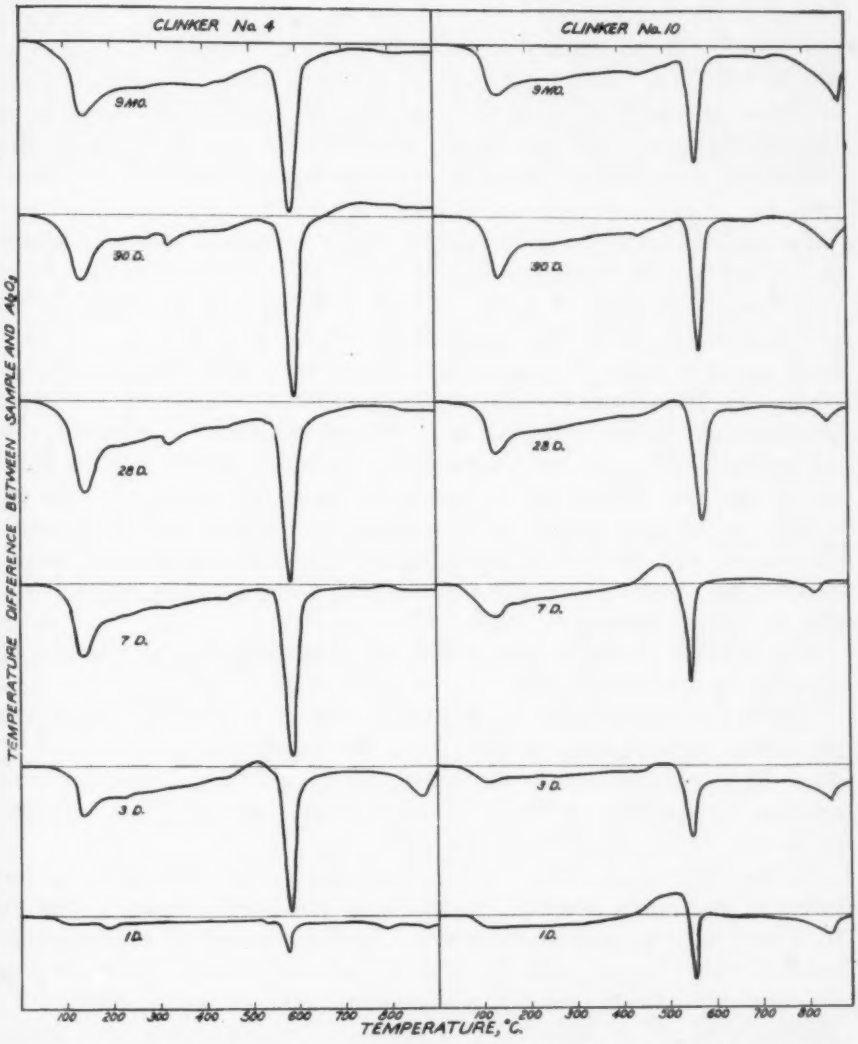

Fig. 5—Thermal analysis curves of cement clinkers No. 4 and 10 at different ages of test without addition of gypsum

days) there was observed a decrease in the amounts of $Ca(OH)_2$ for several clinkers (No. 1, 2 and 4 particularly). The clinker with the highest content of $2CaO \cdot SiO_2$ (No. 10) gave the smallest bulge for $Ca(OH)_2$ at later ages as expected.

A somewhat irregular exothermic bulge was found for most clinkers at temperatures of about 450-500 C at 3 to 90 days. This change was most pronounced for clinkers 5 and 12, particularly at 3 and 7 days.

None of the clinkers except No. 10 showed it at 9 months. Generally associated with the bulge at 450-500 C was an endothermic bulge at 800 to 890 C of various extents. It also was absent from the curves at 9 months except for clinkers 10 and 12. Clinker 4 did not show these thermal changes to any significant extent except at 3 days. No definite correlation was observed between extent of these bulges and the age of test.

Cement clinkers with addition of gypsum—The clinkers with an addition of 1.8 percent SO_3 as gypsum showed in the 1-day test either a well developed dehydration bulge at 150-160 C (clinker 12 in Fig. 6) or an arrest in the curve at this temperature (clinker 1 in Fig. 4). At some later age this bulge, if originally present, eventually disappeared entirely. As it diminished in size the one at 130-140 C generally increased in extent and tended to be the same as that obtained for clinkers without gypsum. However, as illustrated by the results in Fig. 4, no definite correlation existed between the size of the bulge and the age of test. As a rule this thermal change at 9 months was smaller than at 90 days. Clinker 12 was more of an exception to the foregoing generalization, because the addition of gypsum resulted in rather large increases in the size of this endothermic bulge.

The 195-200 C endothermic bulge was eliminated by the addition of gypsum in the 1-day tests.

The characteristic bulge for the solid solution at 210-220 C appeared to various extents and generally was most pronounced at 7 and 28 days. The general pattern was about the same as for clinkers without gypsum, except that the bulges tended to be larger at 7 days for the treated clinkers. Fig. 4 illustrates these changes.

To ascertain more fully the effect of SO_3 on the formation of the hydrous phases in cements, particularly the solid solution, clinkers 10, 1 and 12 (arranged in order of increasing amounts of solid solution formed) were treated with 3.5 and 5.0 percent SO_3. Clinker 12 was also used with 15 percent SO_3, an amount in excess of that required to combine with all the alumina as trisulfate. These additions were made so that solid gypsum would be present at least during the earlier periods, 1, 3 and 7 days. Gypsum is readily detected by two endothermic bulges at about 160 and 190 C for the smaller amounts and at somewhat higher temperatures for the larger amounts.

Clinker 10 with 3.5 percent added SO_3 still contained gypsum at 14 days and the trisulfate at a maximum amount. At 28 days the gypsum was completely combined and the trisulfate was transformed in part to the phase showing the 130-140 C bulge. The solid solution did not appear at any period including 90 days. Clinker 1 with 3.5 percent SO_3 showed no free gypsum at 3 days and the thermal curve contained

bulges at 130-140 C and 150-160 C. The solid solution appeared in slight amount at 28 days only. The trisulfate was still present in large amount at 90 days. Clinker 1 with 5 percent SO_3 contained gypsum at 7 days but not at 14 days, and the solid solution failed to form. Clinker 12 with 5 percent SO_3 contained only the trisulfate at 3 days and the solid solution did not form even at 28 days, the longest test made with

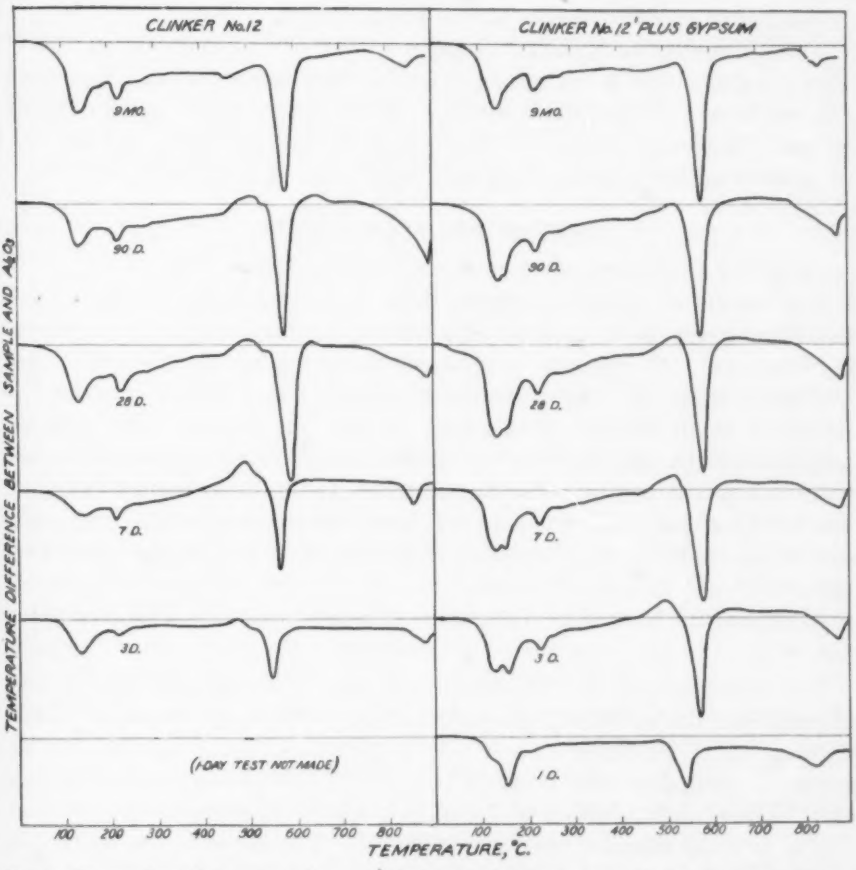

Fig. 6—Thermal analysis curves of cement clinker 12 at different ages of test with and without addition of 1.8 percent SO_3 as gypsum

this addition. The phase showing the 130-140 C bulge was present after 3 days. Addition of 15 percent gypsum to clinker 12 did not repress the formation of the phase giving the bulge at 130-140 C.

Gypsum, when present as the uncombined solid, decreased markedly or eliminated the exothermic bulge at 400-500 C and the endothermic bulge at 800-900 C.

The isometric $3CaO \cdot Al_2O_3 \cdot 6H_2O$ was not found in any sample at any age with any of the additions of gypsum.

A particularly noticeable effect of added gypsum was the marked increase in the extent of the $Ca(OH)_2$ dehydration peak at 1 day compared to the untreated clinkers. The extent of this peak at 7 days approached that of the untreated clinkers. However, at 28 days and later, the gypsum-treated samples compared to those without gypsum showed smaller dehydration bulges for $Ca(OH)_2$.

The characteristic thermal changes at 400-500 C and 800 to 890 C were generally about the same for the clinkers treated with 1.8 percent SO_3 as those without the addition. Some irregularities appeared but no real significance could be attached to such deviations. Clinker 10, as without added gypsum, showed a large bulge at 880 C at 9 months.

INTERPRETATION OF RESULTS

Formation and transformation of the trisulfate form of the sulfoaluminates

The results of thermal analysis show that the trisulfate form of the sulfoaluminates is the initial SO_3-bearing product formed in hydrating cements. Its presence was clearly indicated by the 150-160 C dehydration bulge at 1 day except for cement No. 1 which contained a relatively small amount of this solid at that age of test. The striking and unexpected observation was that the trisulfate transformed to an unknown product apparently characterized by an endothermic bulge at 130-140 C. This unknown solid will be designated phase X. Inasmuch as clinkers without added gypsum also showed an endothermic bulge at 130-140 C not greatly dissimilar to that of the SO_3-treated clinkers it would appear that phase X consists of a solid or solids with and without SO_3.

The formation of the trisulfate from the Al_2O_3-bearing phases and gypsum and the subsequent transformation of this solid to phase X was determined in detail for clinker No. 3 treated with 2.2 percent SO_3 as gypsum. Samples were processed at 1½- to 3-hour intervals starting at 1½ hours after addition of the mixing water. Representative thermal curves of these samples are given in Fig. 7. The curve at the bottom is for the dry cement and shows the two characteristic dehydration bulges of gypsum at 150 and 195 C for the conditions of the present tests. It is clearly shown by Fig. 7 that the bulges for gypsum decrease in area with increasing age of test and finally disappear during the time interval between 14 and 18 hours. Simultaneously the area of the bulge for the trisulfate increased and was at a maximum at 18 hours. Thus Fig. 7 shows graphically the formation of the trisulfate from gypsum.

The transformation of the trisulfate started soon after 18 hours and at 24 hours, as shown by the thermal curve; both phase X and the solid

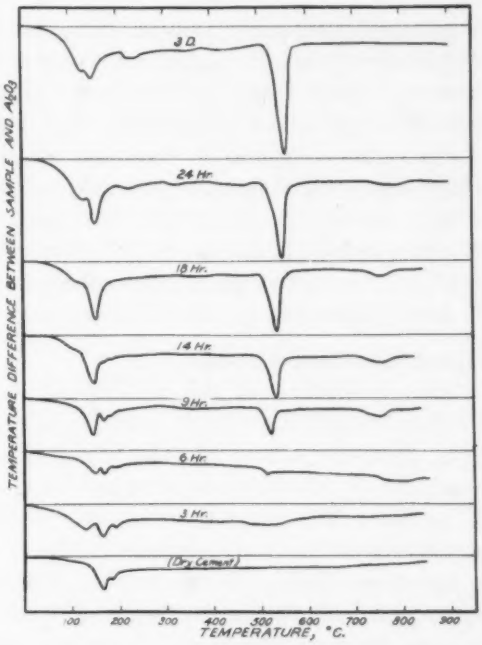

Fig. 7—Thermal analysis curves of cement clinker No. 3 at early ages of test with addition of 2.2 percent SO_3 as gypsum

solution were present. The bulges at 220 C and 330 C although admittedly rather small are nonetheless indicative of the formation of the SO_3-rich solid solution (see Fig. 1, bottom curve). The rate of transformation of the trisulfate was markedly greater in the period between 18 hours and 3 days than at later periods. This rapid transformation during the first 24 hours would account for the small amount of trisulfate found in clinker 1 treated with SO_3 at this age of test. However, it should be noted that the formation of phase X in some clinkers, No. 12 for example, proceeds at a very slow rate. Not until at some time between 90 days and 9 months was the trisulfate substantially transformed to phase X and solid solution in this clinker.

The tests with 3.5, 5.0 and 15 percent additions of SO_3 as gypsum show that the trisulfate is stabilized by excess gypsum, but the formation of the solid solution is prevented. The presence of solid gypsum, of course, implies a saturated solution of this salt. This condition for the stability and instability, respectively, of the two products was shown by studies of the system Na_2O-CaO-Al_2O_3-SO_3-H_2O.[5] However, it now appears that phase X can coexist with the trisulfate in cements in the presence of excess gypsum.

The nonformation of $3CaO \cdot Al_2O_3 \cdot 6H_2O$ in cements

Because the isometric $3CaO \cdot Al_2O_3 \cdot 6H_2O$ was not detected in clinkers containing in excess of 0.05 percent SO_3 it is concluded that this solid

does not form in commercial cements during hydration. The present results show that the amount of SO_3 tolerated in the clinkers before the isometric phase forms is indeed very small. In fact, as shown by Fig. 5, 0.05 percent seems high because the isometric phase appeared as a metastable phase at 28 and 90 days but was not detected at 9 months. This observation substantiates the deduction made previously[11] that the SO_4^{--} inhibits the formation of the isometric phase. In that discussion it was suggested that the SO_4^{--} stabilizes the hexagonal structure of the solid solution; the occurrence of phase X as another potential SO_3-bearing phase was not then known.

Hydrous silica-bearing products

The occurrence of the 400-500 C exothermic and 800-890 C endothermic bulges in cements and also in the synthetic lime-silica gel products, generally at 3 to 28 days, is particularly significant. This apparent similarity is all the more striking because the products in cements just as in the synthetic gels appear to undergo some change with time as evidenced by marked decreases in the thermal bulges at later ages. It is not unreasonable to assume that the gels first formed are metastable with respect to the products characterized by comparatively very slight thermal bulges. Because the product made from solutions of $Ca(OH)_2$ and silicic acid showed only slight thermal changes even at 1 day it appears that this method of preparation gives the stable product at a very early age.

The preparations made from lime and the alumina-silica gel did not show decreases in the extents of the two bulges at later ages. The obvious inference is that alumina stabilizes the gel characterized by the two thermal bulges. In this connection the observation that excess gypsum caused marked reduction or elimination of these bulges in cements suggested the possibility that gypsum withdraws the alumina as the trisulfate sulfoaluminate. It was reasoned that if this was true the addition of excess gypsum to the lime and alumina-silica gel mixture should eliminate or greatly decrease the bulges. Fig. 2 shows that this occurred. This evidence indicates the possibility that alumina may combine with the lime-silica gels in cement. The fact that the synthetic lime-silica gel products without alumina also show the bulges makes it difficult to draw any rigid conclusions from the present data. However, it appears that this problem should be amenable to solution by thermal analysis.

The results on the mixtures containing 0.15 and 0.3N $NaOH$ permit a deduction of the probable concentration of this alkali in the aqueous phase of hydrating cements. The gel in contact with a 0.3N $NaOH$ solution has a composition of $0.2\ Na_2O \cdot CaO \cdot SiO_2 \cdot xH_2O$.[3] A product so

rich in Na_2O cannot form in commercial cements if the lime silicates are completely hydrated because about 4 percent of Na_2O by weight of cement would be required. However, if a cement containing 1 percent of Na_2O should contain the soda in a form nearly completely extracted when the lime silicates are about a quarter to a third hydrated the quaternary gel should form and should be detectable by thermal analysis. Because cement 11 containing 1 percent Na_2O did not show the characteristic bulge for the Na_2O-containing gel it appears unlikely that the concentration of the Na_2O in the aqueous phase reached a value of 0.3N. The products from the 0.15N solution either failed to show, or showed only in a barely discernible extent, the characteristic bulge of the Na_2O-bearing gel. It appears reasonable, and consistent with the results based on the data of the system soda-lime-silica-water, to assume that the aqueous phase of the high Na_2O cement (No. 11) does not contain in excess of 4 to 5 g of Na_2O per liter.

The amount of 4 to 5 g of Na_2O per liter may, therefore, represent the highest concentration present in the aqueous phase of hydrating cements after the anhydrous silicates are largely hydrated. Higher concentrations would prevail only during the earlier periods before the readily soluble alkalies (assuming that the K_2O reacts similar to Na_2O) are adsorbed or combined in the hydrous lime-silica gel.

Information on the composition of the aqueous phase in hardened cements would be of help toward devising means of controlling the "alkali-aggregate" reaction.

SUMMARY

1. This study shows that thermal analysis is an effective and simple method of studying hydrating cements and related hydrous products.

2. The calcium sulfoaluminate, $3CaO \cdot Al_2O_3 \cdot 3CaSO_4 \cdot 31H_2O$, appears to form as a metastable phase only during the first few days in most of the cements studied. The solid solution $3CaO \cdot Al_2O_3$ (xSO_3 (1-x) $(OH)_2 \cdot 12$-$13H_2O$ also appears to form as a metastable phase in some cements of high Al_2O_3:Fe_2O_3 ratio, but not until all gypsum had combined. The stable sulfate phase may be the product characterized by an endothermic bulge at 130-140 C. There is some indication that it may be an Al_2O_3-Fe_2O_3 bearing solid.

3. The isometric tricalcium aluminate hexahydrate does not form in hydrating cements.

4. The results indicate that two different lime-silica gels are formed consecutively in most cements studied. The gel that forms at earlier ages (3 to 28 days) shows a moderate to large irregular exothermic bulge at 400-500 C and generally another large bulge at 800-890 C. The "aged" gel does not show any significant thermal changes and, in this respect, is similar to the lime-silica solution product.

5. Alumina appears to stabilize the synthetic lime-silica gel showing the large thermal changes.

6. Gel prepared in the presence of 0.3N *NaOH* solution and consisting of soda, lime, silica and water shows a characteristic endothermic bulge at 630 C. The aqueous phase of hydrated cements of about 1.0 percent Na_2O content probably does not contain in excess of about 4 to 5 g of Na_2O per liter.

ACKNOWLEDGMENT

The authors are indebted to the Portland Cement Assn. for supplying, complete with analyses, three of the cement clinkers used in this study.

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Although there have been many investigations of volume changes in concrete, the author goes a step further by measuring the stresses in reinforcing steel and concrete due to these volume changes. Data from a pilot project gives information regarding the effect of volume changes on the development of stresses and suggests the advisability of considering these stresses in structural design.

Stresses in Reinforced Concrete Due to Volume Change*

By FRANK R. BEYER†

SYNOPSIS

A study of stresses set up in reinforcing steel and concrete by volume changes. SR-4 gages were attached to the reinforcing steel and after being carefully waterproofed, the test specimens were poured in essentially watertight forms. Strain readings began immediately and continued at varying intervals for three months. A drift test to determine the effect of diminishing battery potential was included to correct for this variable. Charts are presented to show the stresses measured during the tests. The author concludes that with proper precautions the SR-4 strain gage will measure the actual stress induced by volume changes. These stresses, occurring in reinforced concrete in a definite cycle with respect to time, are appreciable and perhaps should be considered in design.

INTRODUCTION

Over the past two or three decades many investigations concerning volume changes of concrete have been made. Almost all have concerned themselves with plain concrete and then generally with volume changes occurring after the concrete is from one to three days old. While these investigations throw much light on the complicated behavior of concrete, it is difficult to translate the information thus obtained into a form usable in structural design and practice. The prime consideration in structural work is not the actual change in the volume of the concrete, but rather the stresses set up by these volume changes. To be of much use in the design of structures, it is necessary to know what stresses are set up in the reinforcing steel and in the concrete due to volume changes resulting from all causes *from the time of pouring.* These volume changes include those due to loading and restraint, ambient conditions and autogenous volume changes.

*Received by the Institute Mar. 18, 1949. Title No. 45-43 is a part of the copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 10, June 1949, *Proceedings* V. 45. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Sept. 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

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While this paper does not pretend to represent a complete solution to the problem indicated above, it is a start in that direction. The experimental work described herein should be considered as a pilot project whose object was two-fold; (1) to devise a reliable method of measuring stress due to volume changes, and (2) to obtain some preliminary information regarding the effect of these volume changes pertaining to the development of stresses. At present the work is being continued by an extensive investigation of the reinforced concrete structural frame and the dynamometer supports of the Engineering Building being constructed at Wayne University. The technique being used is that developed with the aid of the pilot tests described in this paper.

MOUNTING OF GAGES

The SR-4 type of strain gage has vastly extended, and at the same time somewhat simplified, certain phases of experimental stress analysis. Their application to reinforced concrete, while as yet not extensive, has yielded valuable information, not the least of which is the gradual development of a technique by which reliable results may be obtained. The technique successfully employed in the pilot tests and later in the more extensive structural investigation referred to above is essentially that described in a paper by Moncher, Lissner, and Lipson.[1]

Personal experience with "Duco" gages, together with the report of unsatisfactory results given by this type of gage in the paper referred to above, convinced the author that the creep of gages bonded in this manner is excessive and that for reliable long-time strain measurements gages bonded with bakelite cement are necessary. Consequently, type AB-3 gages having a resistance of 119 ohms and a gage factor of 2.08 were employed. The surface of the reinforcing rods was prepared by grinding, filing, and sanding, to remove scale and rust, and to present a surface of the proper roughness for gage application. After removing all grease and dirt from the prepared surfaces and the gages with ethyl alcohol, bakelite cement No. 6035 was applied to the rods and the strain gages were put in place. Pressure was applied to each gage during the baking cycle by a C-clamp through a temperature resistant rubber pad backed by a steel plate. After baking and before the rods had a chance to cool, each gage was covered with several coats of hot Petrosene A wax to prevent them from absorbing moisture from the atmosphere.

At this point the resistance to ground was checked with a vacuum tube voltmeter and found to be in excess of 1000 megohms which, according to A. C. Ruge[2], is essential for continued stability of readings. While a leakage resistance of 1000 megohms is desirable, satisfactory results have been obtained with gages whose leakage resistance is as low as 100 megohms.[1] Next, lead wires of 16 gage, single conductor, Flamenol-

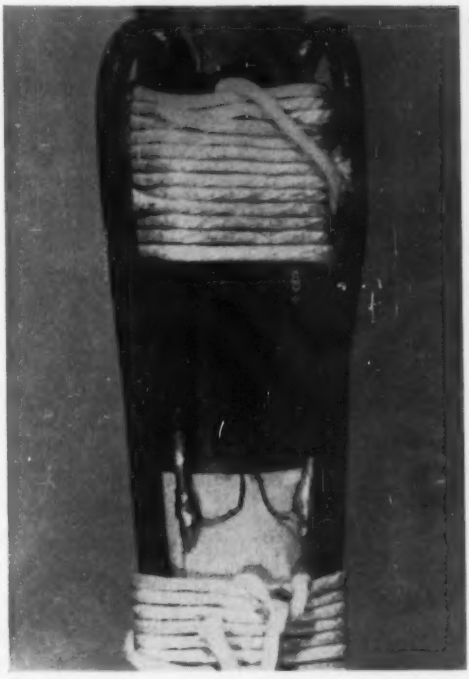

Fig. 1—Lead wires securely tied to rod and soldered to strain gage

insulated wire were securely tied to the rods as shown in Fig. 1 and soldered to the strain gage leads. Then the portion of the rods where strain gages were mounted was completely covered with Petrosene A wax about $\frac{3}{16}$ in. thick. To prevent breaking this wax seal, the lead wires were securely tied to the rods a short distance beyond the wax. Finally, as an added precaution, the entire waxed portion was wrapped with rubber splicing compound. The rods were now ready to be placed in the forms.

CONCRETE SPECIMENS

Four concrete specimens 7 x 7 x 48 in. were made of concrete similar to that to be used for the then projected Engineering Building, each reinforced with one centrally located rod. The general specifications for the concrete were: compressive strength at 28 days—3000 psi, water-cement ratio—6½ gal per sack and slump—3-5 in. The proportions used per cu yd of concrete were: 6 sacks of portland cement, 1350 lbs of sand, 1800 lbs of gravel and 39 gal. of water. Using this mix the results were: slump—5 in., compressive strength at 28 days—3560 psi and modulus of elasticity—3.5 x 10^6 psi.

Two 1-in. round and two 1¼-in. square deformed rods of intermediate grade billet steel, A.S.T.M. Designation A 15-39, each 48 in. long, were

used for reinforcement. One rod was used in each of the four specimens, providing 1.57 and 3.12 percent reinforcement. On each rod two strain gages were mounted near the center, destroying approximately 15 percent of the bond. The forms were well oiled and sealed at the joints with wax. The concrete was poured with the specimens vertical. To prevent loss of moisture, glass plates were placed over the open top ends; but some moisture was lost due to the breaking of the wax seal at the bottom of each form by the excessive pressure of the fresh concrete. A small amount of moisture was also lost through the holes in the forms through which the lead wires were threaded. The forms were removed after two days.

INSTRUMENTATION

Immediately after pouring, an initial strain reading was taken on each gage with the SR-4 Strain Indicator. Readings were taken at various time intervals, at first once each hour and later every two or three hours, then several times a day, and finally only once each day, for a period of about three months.

A few days after pouring, a "drift test" was made to determine the variation of indicator readings due to any cause, but particularly due to discharge of the batteries in the indicator. This precaution was deemed advisable since it was known that trouble had been caused in this manner. Two reinforcing rods, each with two strain gages, were mounted near the test specimens in such a way that they would be undisturbed. These gages were read with the strain indicator at the same time as those in the test specimens. The result of the drift test showed that the indicator reading gradually changed due to battery discharge, indicating an increasing tensile strain (Fig. 2), where actually no change in strain existed. When the batteries finally were replaced readings changed abruptly, indicating some compressive strain, showing that the batteries probably were not fresh at the beginning of the test. This trouble, also mentioned by Tschebotarioff[2], can be easily avoided by the use of a regulated power supply instead of batteries. This power supply is now commercially available and is, for long time tests, indispensable. With the aid of the drift data plotted in Fig. 2 from the average of four gages, it was a simple matter to correct all the test data for indicator drift.

Temperature variation is generally compensated for automatically by connecting an unloaded "dummy" gage in one leg of the strain indicator bridge circuit. The dummy gage ordinarily being at the same temperature as the loaded gage cancels out the effect of any temperature change common to both gages. However, as concrete reacts exothermically, its temperature will be somewhat higher than that of the surrounding atmosphere. This temperature difference, if appreciable, would be

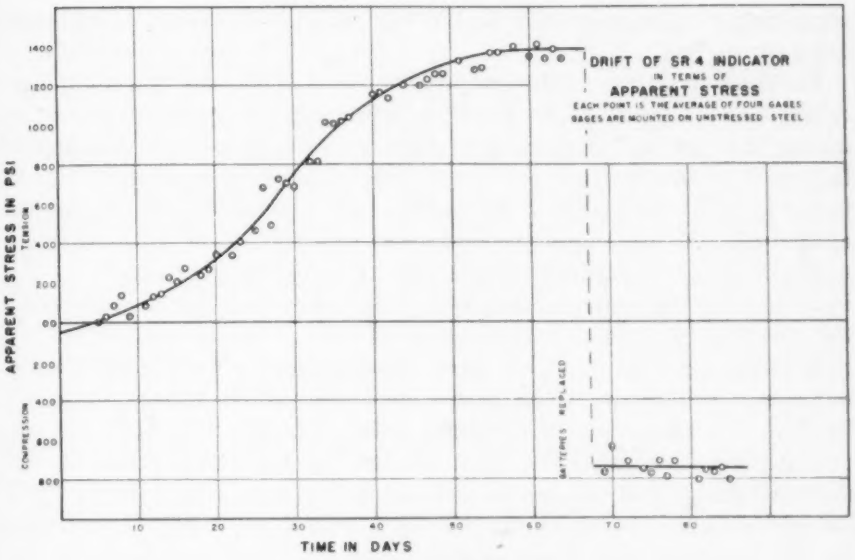

Fig. 2—Indicator drift curve

indicated as a tensile stress in the reinforcing steel. An attempt was made to measure the temperature differential of the concrete and the surrounding atmosphere by means of thermocouples soldered to the reinforcing steel. No definite indications were obtained, signifying that the temperature of the concrete is not appreciably different from that of the surrounding atmosphere in members of this size.

RESULTS

Volume changes which give rise to stresses must obviously be confined to those changes after initial set takes place. Thus the settlement of solids which accompanies the bleeding of water to the top of fresh concrete, while causing an appreciable decrease in volume, can in no way cause stress.

The data presented in Fig. 3 shows the development of stress in the reinforcing steel of the unloaded specimens during the first four days after pouring. No measurable stress was developed until the specimens were about three hours old, which is after the initial set, usually considered to take place within two to three hours. At this point compressive stress was being developed in the steel. This compressive stress increased until the specimens were about ten hours old, final set having taken place in the meantime. The volume change causing this stress is a contraction which according to Davis[3] is attributable in part to the formation of hydrated compounds of smaller volume than the original

components taken separately and in part to the adsorption of moisture films by the gel.

For the next day or so the compressive stress in the steel gradually was relieved and became a tensile stress indicating an increase in volume during this period. According to Davis[3] this is to be expected and is the result of the formation of new hydration solids from the water held in the pores of the hardened concrete as free water and of the expansion of the gel. At later ages the hydration process utilizes all of the free water in the pores and then draws water held by the gel to continue the hydration of the cement particles. This causes the gel to shrink and the concrete to undergo a decrease in volume. This is verified by the data plotted in Fig. 4, which show the development of a continuously increasing compressive stress in the steel after about two days. After 85 days the magnitude of the compressive stress in the steel was approximately 7500 psi. The data plotted for the above figure is the average of the two specimens containing the 1-in. round rods. The data from specimens containing the 1¼-in. square rods resulted in similar curves, giving slightly lower steel stresses. In this case the concrete stresses are nearly double those of the specimens containing the 1-in. round rods, since the percentage of reinforcement is doubled.

While it would have been desirable to plot only the effects of autogenous volume changes, eliminating the effects of ambient conditions as well as those of loading and restraint, it was not possible to do so, as an accurate record of temperature and humidity was not kept. However, since the specimens were in a large room in the basement and the test extended over the summer months, the effect of change in ambient conditions was very nearly negligible.

More important is the effect of the watertightness of the forms. This is demonstrated by Scholer and Dawley[4] who show that the more watertight the form the greater is the amount of expansion during the first day and a half. Consequently, the use of nearly watertight forms, as in the work described herein, results in somewhat greater expansion than ordinarily obtained in practice. As a result, the stresses developed will be higher, though by no means necessarily more detrimental, than those occurring where less watertight forms are used.

The results obtained by Scholer and Dawley are verified by those presented in this paper. Their experiments, in which volume changes of plain concrete were measured with an optical comparator from the time of pouring, were conducted under ambient conditions similar to those existing during this investigation. Comparing their curves[4] showing volume changes of plain concrete to Fig. 4 which shows the stress in the reinforcing steel, a striking similarity is noted. It must be remembered when comparing the two results that Scholer and Dawley measured the

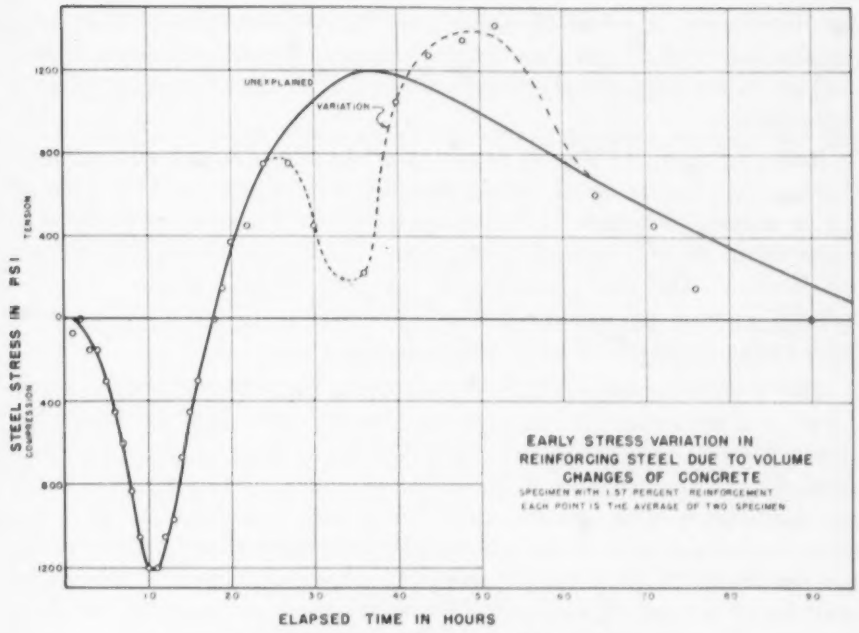

Fig. 3—Stress variation during first four days

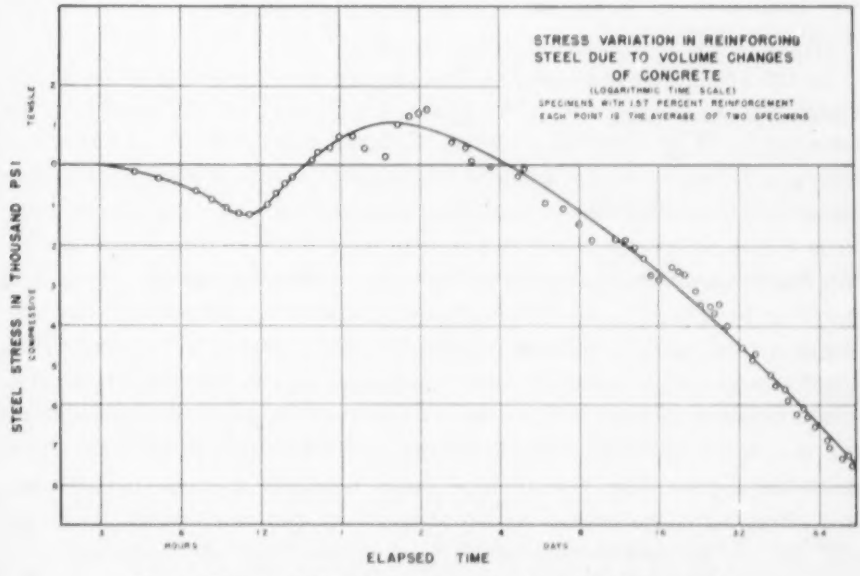

Fig. 4—Stress variation plotted against a logarithmic time scale

volume change, whereas this paper presents the measurement of stress in reinforcing steel due to this volume change. That the volume change and stress are proportional to each other after final set has taken place is self-evident.

Referring again to Fig. 3, to the data obtained the second and third days, a variation in stress, which the author is as yet unable to account for, is noted. Whether this variation is due to the behavior of the concrete or due to other causes is not known. Additional tests must be made to determine the true character of the curve in this region. For the present a curve approximating those of Scholer and Dawley and agreeing with known concrete behavior will be used.

The preceding discussion presents the variation of the steel stress, which may not be the critical stress, and so the problem of determining the concrete stress presents itself. First it is noted that the concrete stress from the causes discussed above is always of the opposite sign to that of the steel stress, while, on the other hand, due to loading, the steel and concrete stresses are of the same sign. The fact that tensile stresses are developed in the concrete from the causes discussed in this paper may be of critical importance, since concrete is exceptionally weak in tension. For example, in the tension face of a reinforced concrete beam the autogenous volume changes will intensify the cracking of the concrete. On the other hand, in the case of a column, the compressive stress existing in the concrete is actually less than that calculated. Whether the magnitude of these concrete stresses is of importance is a matter worth investigation.

In the specimens discussed in this paper there are about 50 sq in. of concrete cross section and 0.785 sq in. of steel, giving 1.57 percent reinforcement. With the steel stress at 85 days being 7500 psi compression, the total load carried by the steel at this time and consequently also by the concrete is 5900 lb. Using this value of load and assuming a uniform stress distribution across the concrete, the concrete stress is 118 psi tension, which is approximately the allowable design stress for concrete in tension. As shrinkage had not ceased, the tensile stress in the concrete would continue to increase at a slowly decreasing rate. Furthermore, it is probable that the actual stress distribution in the concrete is not uniform but is a maximum near the steel. Consequently, the maximum concrete stress resulting from shrinkage is actually considerably higher than the average value obtained above. Comparing this stress with the actual tensile strength of concrete, which is about 400 psi, it is readily apparent that stresses due to shrinkage are appreciable.

SUMMARY OF TEST RESULTS

The following summary is based on data from the unloaded reinforced concrete specimens containing 1-in. round deformed steel rods:

1. No measurable stress was developed until about three hours after pouring, which is some time after initial set had taken place.
2. Between three and ten hours after pouring, shrinkage of the concrete caused a compressive stress in the reinforcing steel while a corresponding tensile stress developed in the concrete. For the specimens concerned, the maximum stresses during this phase were approximately 1200 psi compression in the steel and 20 psi tension on the average in the concrete.
3. During the next day or so the concrete expanded, relieving these stresses. The maximum steel stress in these specimens was about 1200 psi tension with a corresponding compressive stress of about 20 psi in the concrete, at about one and one-half days.
4. After this period, the concrete again contracted, causing an increasing compressive stress in the steel while an increasing tensile stress developed in the concrete. After nearly three months these stresses were 7500 psi compression in the steel and 118 psi tension in the concrete. The rate at which these stresses are developed diminishes with age.

CONCLUSIONS

The SR-4 strain gage may readily be applied with success to investigations involving stresses in reinforced concrete, if proper precautions are taken.

Autogenous volume changes of concrete give rise to a definite stress cycle with respect to time if the concrete is reinforced.

While the concrete actually flows, relieving stress, the stresses determined by this method are the stresses actually present.

These stresses, being ultimately tensile in unloaded reinforced concrete members, are of appreciable magnitude. They tend to intensify the cracking in regions where tensile stresses exist and to relieve existing compressive stresses. Consequently, a consideration of these stresses in design seems advisable.

The shrinkage stresses in concrete become greater as the percentage of reinforcement is increased.

An extensive investigation, including not only stresses due to autogenous volume changes, but also the effects of plastic flow resulting from external loading, might provide quantitative data which would permit more precise and perhaps more economical structural design.

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2. Tschebotarioff, G. P., "Use of Electric Resistivity Strain Gages Over Long Periods of Time," *Proc.*, Society for Experimental Stress Analysis, V. III, No. 2, p. 47.

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Discussion of a paper by Frank R. Beyer:

Stresses in Reinforced Concrete Due to Volume Change*

By A. A. TOPRACTSOGLOU and AUTHOR

By A. A. TOPRACTSOGLOU†

The reliability of electrical strain gage analysis is influenced by the stability of the strain indicating device and of the strain gage itself. The different factors influencing both stabilities were brought to the attention of investigators by G. P. Tschebotarioff whose contribution is mentioned by the author.

However, it must be pointed out that a test does not have to be many weeks or months long to make drift in the strain indicator important. For precise work, when batteries only are used, the drift of the indicator is considerable even in a few days. A drift of about 20 micro-in. in 24 hours must be expected.

Fig. A shows the decay of the strain indicator batteries. An "A" battery must be replaced after eight hours of continuous work with the indicator switch on. "B" batteries seem to last for more than 48 hours.

Fig. B shows the change in the zero reading of a strain gage. The change is due, in this case, entirely to battery decay. The strain gage is that of a dynamometer used in measuring the force applied on a structural member during a recent test. The graph shows that a change of 30 micro-in. took place in 52 hours. For the particular dynamometer used, this corresponds to 1500 lb. It is, therefore, obvious that in any precise test the drift should not be overlooked even if this test lasts for only a few days.

The drift in a strain indicator may be due not only to the change in zero because of battery decay but also to the change in the sensitivity of the instrument.

*ACI JOURNAL, June 1949, *Proc.* V. 45, p. 713. Disc 45-43 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 21, No. 4, Dec. 1949, Part 2, *Proceedings* V. 45.

†Research Assistant, Dept. of Civil Engineering, Lehigh Univ., Bethlehem, Pa.

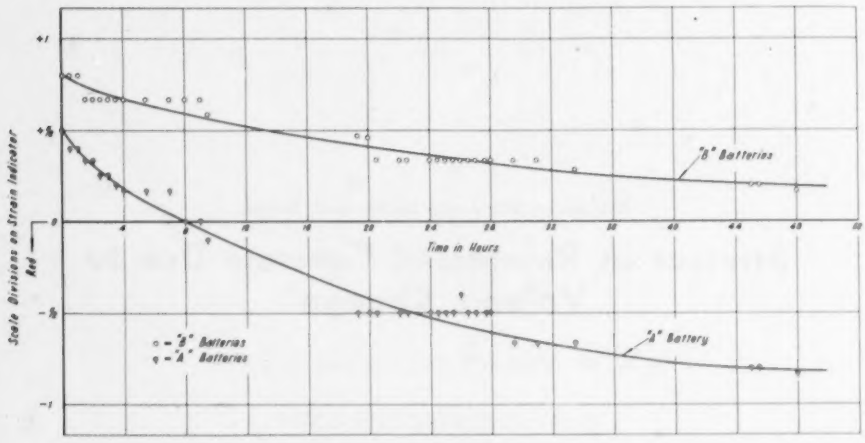

Fig. A—Decay of strain indicator batteries

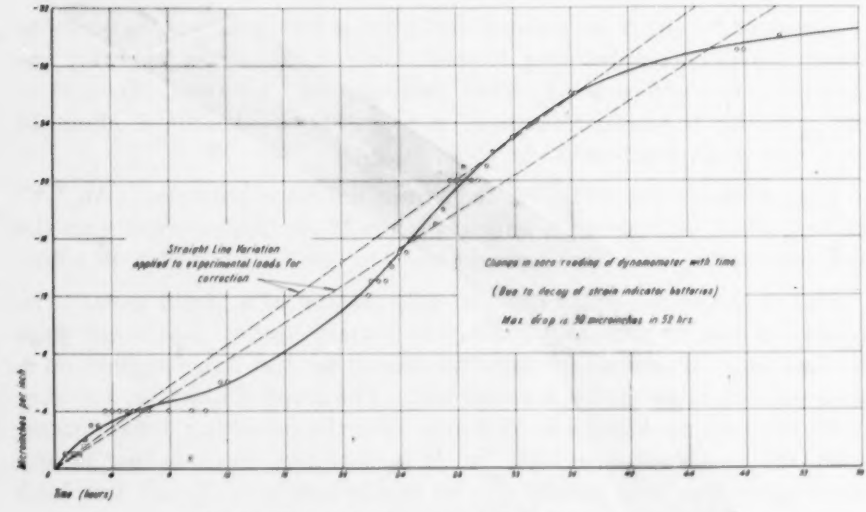
Straight Line Variation
applied to experimental loads for
correction
Change in zero reading of dynamometer with time
(Due to decay of strain indicator batteries)
Max drop is 90 microinches in 52 hrs.
Micrinches per inch
Time (hours)

Fig. B—Change in the zero reading of a strain gage

The "dummy control circuit," as described by G. P. Tschebotarioff, and the other methods given in the same paper are excellent procedure to follow.

The use of a continuous source of power such as an AC-DC converter to power the strain indicator, with the internal batteries left intact for regulation, seems to be the best way to avoid drift in long term tests.

AUTHOR'S CLOSURE

It is most essential, as shown by Mr. Topractsoglou, that every precaution possible be taken to assure the stability of the strain indicating device. The effect of battery decay cannot be ignored in any test involving considerable time.

The indicator drift curve, Fig. 2, shows an apparent change in stress of more than 2100 psi due to battery decay. It is difficult to estimate the actual operating time involved as the batteries were used only a short time for each set of readings. However, decay is quite rapid as readings were taken only once a day for most of the test.

More important, perhaps, than improving indicator stability is the establishment of a reference circuit such as the drift test circuit mentioned in the paper. By means of such a device one at least knows what corrections to make, no matter what the cause of the drift may be. It is the author's opinion that, in any case, a reference circuit is indispensable for accurate results.

To meet a need for authoritative information on floor construction, Mr. Klock describes construction practices, for both bonded and monolithic floor finishes, which give a superior floor surface.

Monolithic and Bonded Floor Finishes*

By MORGAN B. KLOCK†

SYNOPSIS

The author discusses the design and construction of two types of floor finish. A floor capable of withstanding heavy wear, either bonded or monolithic finish, can be obtained by using a stiff mix of sound, well-graded materials which is thoroughly compacted with a power float. Hard coarse and fine aggregates are recommended for the greatest resistance to wear.

INTRODUCTION

Concrete floors capable of withstanding heavy use with minimum maintenance require exacting specifications and strict adherence to them during construction. The construction methods described in this paper have been adopted by the Eastman Kodak Co. based on years of experience in building, using and maintaining several million square feet of finished concrete floors.

Two types of floors have been built: a *monolithic finish slab*, in which the finish course is placed while the base is still plastic; and a *bonded finish* in which the finish course is applied some time after the base has hardened.

In reinforced concrete structures the bonded finish has been preferred as it can be laid as one of the last items on the job not having to endure the marring and hard use in the construction of floors above. Monolithic finish floors have been used generally for slabs laid on the ground.

In both bonded and monolithic finishes a very stiff mix of sound, well-graded materials, thoroughly compacted during floating, will produce a floor superior to that obtained by other methods, and at a reasonable cost.

There appears to be no practical difference between different makes of portland cement if they conform to the A.S.T.M. specification for the

*Received by the Institute, Sept. 12, 1947. Title No. 45-44 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 10, June 1949, *Proceedings* V. 45. Separate prints are available at 35 cents each. **Discussion** of this paper (copies in **triplicate**) should reach the Institute not later than Sept. 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

†Member American Concrete Institute, Structural Engineer, Eastman Kodak Co., Rochester, N. Y.

type used. Air-entraining cement provides a slightly fatter mix which makes placement easier. Lumnite cement improves corrosion resistance somewhat, but finishing is more difficult because of the heat generated and the consequent rapid drying during the first 10 to 24 hours.

The harder the sand, the greater will be the resistance to wear. Silica sand for finish aggregate is shown by test to be well worth its extra cost.* If silica sand cannot be found the hardest sand available should be used. Coarse aggregate, as well, should be the toughest material available.

MONOLITHIC FINISHED FLOORS

For soil bearing slabs a properly constructed monolithic floor finish will give an excellent wearing surface. Inferior aggregate (from the standpoint of wear) is used in the base, and tougher aggregate for the exposed finished surface which must withstand abrasion and possible abuse. Obviously, the tougher the aggregate for the exposed finished surface and the closer the control of the grading, mix and application, the better the resulting finish will be. Such a monolithic finish is less expensive than a bonded finish applied later.

Preparation of subgrade

The nature of the subgrade should be known. Inferior soil such as clays, mucks, etc. should be drained and treated to increase their suitability. Fill material should be carefully selected and placed in thin level layers not over 6 in. thick and thoroughly compacted. Over the prepared stabilized subgrade a 3-in. leveling course of run-of-bank gravel, slag, stone dust or other suitable material should be placed and thoroughly compacted by rolling or tamping. All fill material should pass a 1-in. screen.

Design

The thickness of the slab is determined by subsoil conditions and the type and nature of loading. A minimum reinforcement of 6 x 6-in. mesh made of 6-gage wire should be used. Additional reinforcement should be provided to offset settlement of subgrade adjacent to walls and other excavated areas. If reinforcing steel is omitted, the base slab should be increased in thickness by a minimum of 20 percent depending on the finished subsoil conditions. The rough floor slab should be at least 5 in. thick, constructed of concrete designed for a minimum compressive strength of 3500 psi.

Aggregates

The fine aggregate should have clean hard durable grains of low water absorption, uniformly graded to the composition shown in Table 1.

*Fairchild, L. F., "Concrete in Factory Construction," ACI JOURNAL Nov.-Dec. 1934, *Proc.* V. 31, pp. 157-160. See also "Concrete Wearing Surface for Floors," Report of Committee 804, ACI JOURNAL, Sept. 1938, *Proc.* V. 35, p. 27.

TABLE 1—AGGREGATE GRADING FOR MONOLITHIC FLOOR FINISH

Fine aggregate		Coarse aggregate	
Sieve size	Percent retained	Sieve size	Percent retained
No. 4	0	⅜-in.	0
No. 50	85–95	No. 4	95–100
No. 100	95–100		

The coarse aggregate should be clean hard and durable and free from soft, thin and deleterious particles. The abrasion loss determined by A.S.T.M. Specification C 131-46 should not be more than 25 percent by weight. The coarse aggregate grading should conform with that shown in Table 1.

Proportions

The proportion of fine and coarse aggregate should be between 40 to 60 percent fine aggregate with a resulting fineness modulus of 3.75 to 4.50.

The floor finish is mixed in the proportion of one sack of portland cement to 3.0 to 3.5 cu ft of mixed aggregate, the selection being based upon the characteristics of the aggregate. The amount of combined aggregate per sack of cement should be such that the cement paste slightly overfills the voids in the aggregate so that enough paste will be brought to the surface to produce a smooth troweled finish. There should be no surplus of paste on the surface.

Control of mixing water is most important. For mechanical finish, water is held constant between 2½ to 3 gal. per sack of cement, correction being made for the moisture in the aggregate. Fig. 1 and 2 show the low-slump concrete produced.

Construction technique

The rough slab is placed as dry as can be worked or a maximum of 2-in. slump. It is then screeded to ¼ in. below the finished floor elevation. As soon as practicable, but not over 45 minutes after screeding the

Fig. 1—The stiff mix shows no appreciable slump

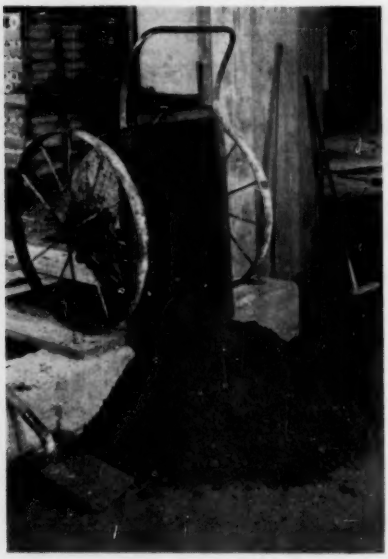

Fig. 2—The concrete finish being placed is one part cement to 3.1 parts of sand by volume with 3.5 gal. of water per bag of cement

rough slab, a $\frac{3}{8}$ to $\frac{1}{2}$-in. layer of topping is uniformly spread over the surface and screeded to the proper grade. When the surface can support the mechanical equipment, it is thoroughly compacted and floated with a heavy rotary float (Fig. 3). It is then troweled at the proper period to a smooth dense finish. The finished floor is kept continuously wet for seven days after it is laid, during which time no heavy traffic is allowed over the new surface.

BONDED CONCRETE FLOOR FINISH

Base slab

The structural slab may be prepared for the topping by brushing during the setting period with a stiff wire brush once over in one direction so as to leave ridges in the surface. If this is done at just the right stage during the setting of the concrete, a satisfactory surface for bonding the finish will be obtained. Where there is laitance on the surface, or where the slab sets before it is brushed, roughening of the slab will be required before placing the topping.

The base slab is thoroughly cured for a period of at least 10 days before the finish is applied. During this interval the base must be protected from grease, paint and other foreign substances. Immediately prior to placing the finished topping, the base course must be thoroughly wetted and cleaned by scrubbing with clean water and a stiff brush. Any foreign substance which cannot be removed by scrubbing is chipped away with picks and the base left thoroughly roughened.

Fig. 3—After tamping and screeding operations, the power float is applied, followed by one or two treatments of the hand trowel, according to the degree of smoothness required

All surplus water is removed and a wash of cement mortar applied. This wash consists of one part cement, one part concrete sand and 4½ gal. of water per bag of cement, and is thoroughly brushed onto the slab with a stiff wire brush. The concrete finish is applied immediately after the mortar wash.

Aggregate and mix proportions

The best aggregate for a bonded finish is a uniformly graded quartz, granite or dolomite. It should be made up of coarse sand and pea gravel, such that 100 percent passes a ⅜-in. mesh screen and not more than 5 percent passes a No. 50 screen, and combined in the proportions yielding the minimum volume of voids. The fineness modulus should be between 3.9 and 4.3. The mix should be proportioned to suit the mixed aggregates, so that the cement-and-water paste overfills the voids in the aggregate by about 5 percent. The mix should be about 1 to 3.1 by dry volume or 1 to 3.8 by weight. The water content should not exceed 3.5 gal. per bag of cement, and the cement content will then be between 8.5 and 9.5 bags per cu yd depending on the void content of the sand. All finish should be mixed for not less than 3 minutes, preferably in a revolving-blade mixer. The resulting material, ready for application, has no slump and contains just enough moisture so that it can be packed into a ball in the hands.

Placing and finishing

Power floating should start not more than ½ hour after mixing; retempering of mixed material should not be permitted. The finish is spread

approximately ¼ in. higher than the elevation of the finish floor, tamped thoroughly, and then screeded to the proper elevation. Special care is taken to see that all lumps of finish material are broken and distributed. A power float is then applied, followed by hand troweling to eliminate all remaining inequalities and ridges. In general, three overall trowelings will be required; in wet spots a fourth troweling may be necessary. No dry cement, sand, or water is added during the finishing process.

Curing

After the finish is 24 hours old, the surface is thoroughly saturated with water and kept continuously wet for a period of seven days.

The topping may be cured by spraying on an approved curing compound, applied according to manufacturer's recommendations, as soon as the surface is dry enough to take it.

All heavy traffic should be kept off this finish for ten days.

Burnishing

After the floor is four weeks old and completely dried, it is burnished to remove the excess of fine particles from the surface. The floor is sprinkled lightly with water, covered with a thin layer (1/16 in.) of sand blast sand, then scrubbed with a wire brush attachment on a power float. The float should be moved slowly and steadily, at the rate of 400 to 500 sq ft per hour. Care must be taken not to allow the brush to revolve in one spot for more than a few seconds, in order not to expose the aggregate. Immediately following the scrubbing the loose material is removed and the floor washed.

CONCLUSION

A floor finish placed with the use of a power float is definitely superior to one placed by any other method. Using a power float, the mix can be so dry that it is not workable by any other means. This dry mix contains an absolute minimum of water which results in low shrinkage in the concrete. The power float with the dry mix produces a finish with a maximum of coarse aggregate at the surface where it is needed to withstand wear. Floors of this type exhibit a small initial wear which is actually the removal of the skin coat of cement paste which lubricates the trowel in hand finishing. Burnishing removes this skin coat and eliminates this initial wear and the dust which accompanies it.

While power equipment permits the use of a stiffer, drier mix it does not relieve the engineer or contractor from the necessity for sound materials and good workmanship.

Discussion of a paper by Morgan B. Klock:

Monolithic and Bonded Floor Finishes*

By HENRY COMACK, ANDREW EBERHARDT and AUTHOR

By HENRY COMACK†

Interest is expressed in the author's application of relatively dry concrete mixes for floors. Based upon laboratory investigations[1] of dry tamped concrete the writer advances the prediction that, because dry tamped concrete eliminates many evils, produces higher strengths and durabilities and uses less cement, many types of structures will be composed of this material. It is suggested that dry tamped concrete bears the same relationship to vibrated low-slump concrete as the latter does to normal concrete. Progress in each case can be attributed to the application of machinery as an aid in placement, permitting stiffer mixes. The principle of vibration reduced the cement paste content. Future use of dry tamping will further reduce cement paste. Confidence is expressed in the manufacturers' ability to develop the necessary field tamping equipment, thus confirming the conclusions found by the writer in his laboratory investigations.[1] The Germans recognized and applied such principles in the Autobahnen.[2]

The advantages of dry tamped concrete are as follows: (1) greater strength and durability than comparable slump concrete; (2) savings of cement; (3) reduction in laitance, bleeding and segregation; (4) dense, hard formation; (5) ability to be conveyed without agitation; and (6) reduction of initial shrinkage.

It is noted that Mr. Klock designed mixes using low water-cement ratios, between 2½ and 3 gal. per sack of cement. Based upon studies, it is suggested that strength and durability are not primarily due to the water-cement ratio alone. They are also dependent upon the density.[3] Higher densities than normal concrete are attained by reduced cement paste content made possible by machine tamping. Water-cement ratios apply only when the consistency of the mix and the

*ACI Journal, June 1949, *Proc.* V. 45, p. 725. Disc. 45-44 is a part of copyrighted Journal of the American Concrete Institute, V. 21, No. 4, Dec. 1949, Part 2, *Proceedings* V. 45.
†Civil Engineer, N. Y. State Dept. of Public Works, Flushing, N. Y.

method of compaction remain constant.[3] In other words, if the established water-cement ratios commensurate with the desired strength are used, considerable reduction of the cement factor is achieved in dry tamped concrete. In addition, reduction of the cement paste in concrete reduces the effects of elemental attacks, thereby suggesting its use as a road building material.

Contrary to popular belief, dry tamped concrete mixes are not ordinary concrete mixes with reduced water-cement ratios. They require special mix designs having special optimum sand-aggregate ratios. The water-cement ratio is determined from Duff Abrams' tables, as in normal concrete, but the cement paste content is dependent upon the compaction method that is applied to the test specimens. Studies indicated that within the range of no-slump mixes the workability varied considerably.

Mr. Klock's use of the power float for finishing and compacting is very interesting. Perhaps, it is well to describe the tests made by the writer with various means of compaction. It was found that tamping equipment for the dry mixes are most satisfactory with specified characteristics. To illustrate, 6 x 12-in. cylinders were molded with the following methods of compaction: (1) gradual application of high static pressures; (2) hand-tamping with a flat-edged steel rod; (3) pneumatic hammer at 1100 short strokes per minute and (4) electrical hammer at 2200 short strokes per minute.

The third and fourth method produced the stronger, more dense and durable specimens. The surface appearance resembled that of the comparable normal concrete specimens. The electrical method at 2200 short power strokes per minute gave the best results. For a floor finish, it is suggested that a combination of floating and high frequency tamping may improve the texture, finish and permit thicker lifts. For a highway pavement, a combination of screeding and tamping may be applied.

In conclusion, the writer visualizes future structures, particularly roads, of dry tamped concrete as a logical trend. It is anticipated that equipment manufacturers will cooperate with concrete engineers to develop the machinery that will give the optimum results; and it is hoped that leaders in concrete research and organizations will pave the way by sponsoring further research to accelerate this development.

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By ANDREW EBERHARDT*

The valuable description of good practice in producing concrete floor finishes by Mr. Klock covers all the basic requirements for a lasting surface: preparation of subfloor, proper aggregates, water-cement ratio, power floating, etc. However, there is no mention of shrinkage cracks which usually mar every concrete structure.

It is realized that a monolithic finish placed on top of a structural slab will crack wherever the slab happens to crack. Therefore, not much can be done here to protect the finish.

If, however, a similar thin finish layer is poured on top of a 5-in. soil bearing slab (discussed by the author) effective control joints can be provided in the form of tooled grooves deep enough to weaken the bearing slab. To make such joints fully effective half or all of the reinforcement has to be cut at this point. If the grooves are objectionable they can be filled after shrinkage takes place.

It would be interesting to hear from the author on the extent of cracking in the floors built according to his specifications and on the use of control joints.

And one more question: What is the thickness of a bonded finish he recommends?

AUTHOR'S CLOSURE

In answer to Mr. Eberhardt, the bonded finish which was discussed in the original article is intended to be 1 in. thick. This naturally will vary with the unevenness of the rough slab. When the placing of finish is delayed for a prolonged period we have noticed a sag in the flat slab panels which we overcome with finish material. This may add as much as $\frac{1}{4}$ in. to $\frac{3}{8}$ in. to the thickness of the finish in the center of the panel.

Shrinkage cracks do not seem to occur with the dry mix except over cracks which develop in the base slab. We have put wire mesh in the finish over these cracks as well as over construction joints with fair success. I suspect that where there is a tendency for differential movement that the cracks will ultimately come through.

We have used a tooled joint over construction joints to control the crack and it has worked satisfactorily. The control joint should be filled to avoid the accumulation of dirt and also the joint is a point of

*Chicago, Ill.

weakness under heavy trucking if not filled. We have never found a completely satisfactory filler compound but find asphalt mastic fairly easy to maintain.

I agree with Mr. Comack that the dry tamped mix gives different results from normal consistencies. It is impossible to make sample cylinders using the ordinary method of rodding. Our original tests were based on variable gradings of sand with a fineness modulus of approximately 3.5 to 4.2. By direct comparison and elimination of the poorest samples after each test, we arrived at the present recommendation. Wear resistance increased as the fineness modulus increased up to a point where the mix was so coarse that troweling became difficult. By the same method, samples with less cement but the same sand grading indicated less resistance to abrasion.

Various compacting methods were experimented with, including a vibrator attached to a tamper, a small air hammer attached to a tamper and various types of tampers including steel grating, solid steel plate and solid wood, all approximately 12 in. square.

At present we are using power floats equipped with hammers which give us a final compaction along with the floating. This seems to be the most satisfactory method so far attempted. The manufacturer of the float is still experimenting with the weight of the hammer and number of impacts.

One objection to high frequency vibration for compacting was that it seemed to bring more of the fines to the surface than the hammers on the power float. The fines at the surface are not wear resistant and there is a tendency toward crazing of the finished surface.

Difficult site conditions influenced the selection of

Pumped Concrete for Enders Dam Spillway*

By C. O. CRANE†

SYNOPSIS

A detailed description is given of the use of pumped concrete in the construction of the concrete spillway for Enders Dam. The author calls attention to the many features of pumped concrete that are of interest and value in considering this method of concrete placement.

INTRODUCTION

Enders Dam, one of the units of the Missouri River Basin Project being built by the Bureau of Reclamation, is on the Frenchman River, about two miles south of Enders, Neb. The construction of the concrete spillway for this dam, designed for a maximum flood discharge of 200,000 cfs and involving approximately 42,000 cu yd of concrete and over 6 million lb of embedded and nonembedded metalwork, presented some site conditions that made pumped concrete a practicable means of placement. This spillway involved over 5 acres of concrete floor slabs with thicknesses ranging from 15 in. to 4½ ft, and more than 1400 linear ft of heavy counterforted walls, the larger portion of which extended roughly 50 ft above subgrade. Nebraska state highway No. 61 eventually will be diverted over the adjacent earth dam and across the spillway on a 369-ft bridge supported by intermediate concrete piers. Six 50 x 30-ft steel drum gates, each weighing 161 tons, will operate under the bridge and between the piers. The deck elevation of the reinforced concrete "T" beam type bridge is 49 ft above the foundation. Some distance downstream from the gate structure the spillway floor curves downward on a 200-ft radius curve followed by a 2:1 slope which continues to the floor of the stilling pool. The total length of the curve and slope is 172 ft with a drop in elevation of 72 ft.

On this inclined portion of the structure, the width of the channel increases to 400 ft, making an area of about 58,000 sq ft inclusive of wall

*Received by the Institute Apr. 4, 1949. Title No. 45-45 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 20, No. 10, June 1949, *Proceedings* V. 45. Separate prints are available at 35 cents each. **Discussion** (copes in **triplicate**) should reach the Institute not later than Sept. 1, 1949. Address 7400 Second Boulevard, Detroit 2, Mich.

†Member American Concrete Institute, Engineer, Bureau of Reclamation, Enders Dam, Enders, Neb.

footings that is partially or wholly inaccessible to conventional concrete mixing and placing equipment without the erection of unusually heavy scaffolding. This feature, together with the high walls and high bridge structure, all above the reach of any available crane and bucket on the job, caused the contractor to consider the use of a pump for delivering the concrete to these rather inaccessible locations. The concrete specifications for Enders Dam established the maximum size of the aggregate at 1½ in. for all mixes and a concrete slump not in excess of 4 in., two features which set the quality of the concrete well within the limits of pumped concrete.

After considerable study and deliberation on the use of pumped concrete as compared to other methods of placing, such as buggies, overhead cable, tower hoist, cranes, etc., the project manager for the contractor obtained a double concrete-pumping machine together with a pugmill type remixer and an adequate quantity of 8-in. steel pipe and fittings. This equipment arrived early in the spring of 1948 and was used intermittently on various concrete placements throughout the entire season (Fig. 1 and 2).

While this method of placing concrete has not proved to be a solution for all problems pertinent to the work, experience has shown that pumped concrete has several definite advantages. The experiences on the Enders work point out the advantages of pumped concrete where it is adaptable as well as indicate certain possible exceptions to its employment in concrete work.

CONCRETE CHARACTERISTICS

The concrete was mixed in a paving mixer located on a terrace above the pumping machine so that the paver discharged directly into the pugmill of the pump. Concrete containing from 5 to 5½ sacks of cement per cu yd and a sand-gravel ratio of 33 percent by weight of the total aggregate was used. Between 3 and 5 percent of air was entrained in the concrete. All aggregates, including sand, were carefully graded, with limitations on undersize in each of the coarse aggregate sizes. Two sizes of coarse aggregate were processed, ¾-in. and 1½-in. maximum. Cement and aggregates were dry batched and trucked to the mixer skip. Through this control, only minor fluctuations in the consistency of the concrete were possible. Once a pumpable mix was established, difficulties thereafter encountered rarely were attributable to the concrete.

PIPELINE DISTRIBUTION

The pumping machine consisted of two pumps which fed through a siamese connection into a single 8-in. O. D. line. A total of 1000 ft of line, consisting of 10-ft straight lengths and a few 11½°, 22½°, 45°, and

Fig. 1—General view of concrete pumping operations for a section of the right spillway channel wall in the stilling pool area. The pipeline from the pump can be seen leading to a point above the form. This form on the sloping portion of the spillway was, like many other similar sections, located beyond the reach of the crane. Here the concrete was pumped against a head of 33 ft. For quick temporary setups such as this, a crane and bucket was used to transport the concrete from the mixer to the pump, rather than locating the mixer on a terrace of ground adjacent to the pump.

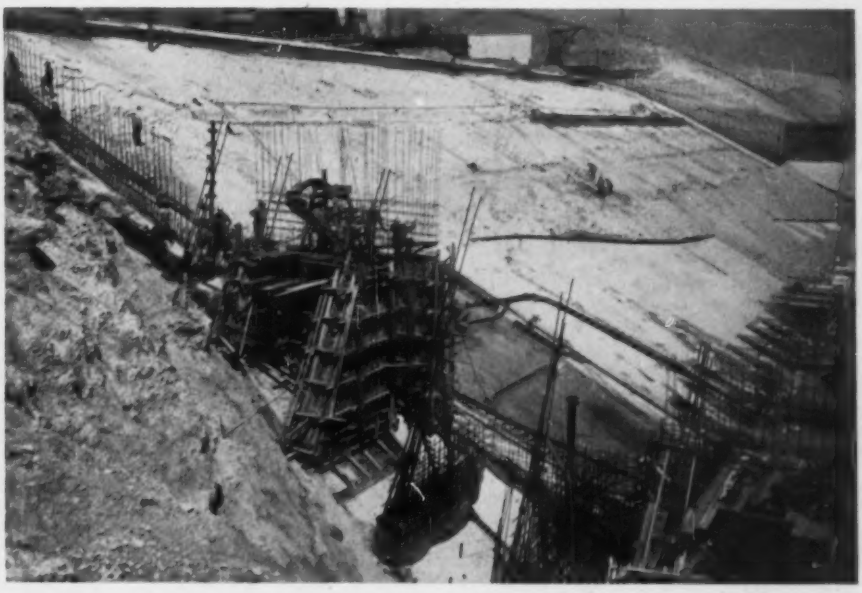

Fig. 2—Rear view of the same form shown in Fig. 1. The pipe line, discharging into a metal chute which was adjustable in length, can be seen at the top of the form. Four rubber tremies were hung in the form to receive the concrete from the chute. Pumping was stopped while the chute was moved from one tremie to the next. This arrangement, typical of that used on counterforted walls, resulted in a good job with little or no segregation of the concrete at the point of placement.

90° ells, were obtained for use beyond the siamese connection. A discharge fitting for the end of the pipe and a few special sections were among the supplies also furnished. A "go-devil" for cleaning the pipe, together with its entry tube and fittings for an air line connection, was included with the equipment. The 10-ft sections of pipe weighed 230 lb and required four men to handle.

The rate of laying the pipe was, of course, variable and depended on the terrain, distance the pipe was carried, nature of pipe supports, and the caliber of the men. When the pipe was laid directly on the ground or on sacks of sand, and when other conditions were favorable, as much as 150 linear ft per hour was laid by four men. However, when it was necessary to extend the pipeline to the top of a wall form, the work was increased considerably. Substantial bridging was required to support the line and more men were required to move the units up the slope and hold them in place while the ends were joined with the snap-on toggle connection. Because of the pulsating action of the pipeline, resulting from the reciprocating pump strokes, it was necessary to support the pipe entirely separate from the forms. This required extra longitudinal bracing and scaffolding for the pipe supports. Fig. 3 shows the bridgework erected for one of the wall pours on which the top of the form was 28 ft above the elevation of the pump outlet.

In starting a concrete placing operation, it was common practice to throw a bucket or two of water into the pump and follow this by a grout batch as a means of lubricating the pipe and minimizing the possibility of plugging the line at the outset of the work. The water was caught in pails at the discharge end of the line and wasted outside the form. Where the concrete was placed in lifts, the grout served to improve the bond to the old concrete after a suitable joint cleanup had been made. A half-inch mortar layer was required by the specifications, and in many runs the grout batch served to fulfill this requirement. However, for floor slabs and footings the grout primer batch was superfluous.

Complications were encountered on large construction joints, such as those on the crest structure involving an area roughly 51 x 58 ft, where it was desirable to place the mortar layer only a short time in advance of the placing operations. After the pumping line had been filled with concrete, it was not practicable to inject a grout batch for use on the remaining portion of these large construction joints. In attempting to pump such an intermediary batch it was found that the grout intermixed with the concrete ahead of and back of it, which made the grout less effective as a joint bonding material.

The minimum mixing time per batch for the concrete was limited by the specifications to not less than 1½ minutes. The paver therefore discharged 1⅓ cu yd of concrete about every 1¾ minutes which was approximately equal to the capacity of the pump for the concrete mix used. A 500- to 550-ft line has a volume of about 6 cu yd. The pugmill, which was maintained relatively full to prevent a concentration of rock tailings from the mixer-discharge from dropping directly into the pump and plugging the line, contained approximately two batches most of the time. It can be seen, therefore, that appreciable time elapsed

Fig. 3—Delivery pipe line and scaffolding for a wall form 28 ft above the pump discharge.

between mixing and final placement. Changes required in the mix were necessarily delayed several batches. This sizable volume of mixed concrete on hand complicated the termination of a placing operation since it became necessary to estimate from six to seven batches (8 to 9 cu yd) in advance to stop the batch plant and trucks in sufficient time to prevent a substantial overrun of concrete. However, as experience was gained it was possible to make relatively accurate estimates and overruns or shortages, which were made up by extra batches delivered by wheelbarrow or other hand work, were held to a minimum.

Difficulty was encountered in placing slabs and footings after the concrete left the line because it was necessary to chute a large amount of the concrete into final place and to flow some of it by vibration, both of these operations being recognizably conducive to segregation. To minimize segregation of the concrete beyond the pipeline, increased emphasis was placed on a proper set-up, elevation of the pipe discharge, and the location and direction of the pipe so that as the work progressed lengths of pipe could be removed and the concrete placed as nearly as possible in its final position. On heavily-reinforced footings 4 to 6 ft thick, which required several hours to complete, the pipeline was supported on metal legs which were cut off a few inches below concrete grade after the pipe had been dismantled and before the concrete was screeded for finishing. This eliminated the trouble resulting from the use of timber supports which often stuck fast in the partially set concrete. Most of the wall panels were about 35 ft long and had two counterforts for each section. The pipeline discharged into a short, funnel-shaped chute and subsequently down rubber tremies hanging in the form. As a result little or no segregation in these placements occurred and the operation was considered very satisfactory.

Occasional plugging of the line was unavoidable and occurred seemingly without variation in the mix. Line plugs were common on hot days when over 400 ft of pipe were being used or where appreciable

vertical lifts, 25 ft or more, were employed. By covering the line with burlap and keeping it continuously wet plugging was held to a minimum. This procedure also maintained the concrete temperature below the maximum of 90 F as required by the specifications. A line plug generally delayed the work approximately an hour, and in a few instances where cold joints developed, the entire remaining part of the placement was deferred until the following day and a joint cleanup made before continuing the work. Disconnecting the pipe sections and handling the 10-ft units, which weighed about 600 lb when full of concrete, required appreciable time and was expensive. During hot weather, when the concrete set quickly, as many as 20 men were used to locate and release a single plug. Often it was necessary to disconnect the pipeline above the reinforcing steel. In this operation some pipe sections full of concrete occasionally fell on the reinforcing steel which was already overloaded by workmen lifting on the pipe. Adding more support to the reinforcing steel to withstand these loads, as well as straightening bent bars, increased the cost of pumping concrete.

At the completion of a run there were two ways to force the go-devil through the line to remove the concrete and clean out the pipe—by water pressure, or by air pressure. Since air lines were readily available and the use of water necessitated special care at the discharge end, air pressure was used. The 80 psi air pressure available provided ample force to move the concrete under almost any condition or line length, except direct line plugs. However, as the go-devil approached the end of the line the force required to move the remaining concrete became progressively less and ultimately near the end the air could be shut off and the expansion of the compressed air in the pipe would move the remaining concrete from the line, often at an increased velocity. Although the end of the line was directed downward, some of the rocks from the mix were occasionally blasted on the finished concrete surface causing extra finishing work.

CHARACTERISTICS OF PUMPABLE CONCRETE

No difficulty was experienced on the Enders work by loss of slump of the concrete while in transit through the line. Control slumps were taken at the mixer, and since any concrete that could be pumped could be placed and compacted by vibration, slump loss in transit was not of consequential importance. To say that no loss in slump occurs during transit would, however, be erroneous. The drop in slump that develops during line transit is variable and depends on temperature, rate of pumping, length of line and absorption of aggregate. Published information on pumped concrete indicates an average loss in slump of 1-in. drop per 1000 ft of line. Experience with the Enders concrete closely

corroborated this figure. Since no high slumps were used no segregation occurred in the line while the concrete was in transit and the concrete as discharged was in excellent condition. In fact, as a result of using an air-entraining agent, subjecting the concrete to the line pressure, and the troweling action caused by the concrete moving along the pipe, the concrete appeared more cohesive and plastic at the point of discharge than it did when delivered fresh from the mixer.

The use of a pump provides excellent control of concrete consistency. Only workable, plastic mixes can be handled readily. If a mix is dry, undersanded, or wet, it cannot be satisfactorily pumped. Pumpable concrete requires enough sand and cement paste to form a plastic mixture and prevent interference among the coarse aggregate pieces. Observations on the Enders work indicated that experienced concrete technicians could provide more reliable information on mobility and flowability of a concrete mix from visual inspection of the cohesiveness, plasticity, and workability characteristics than could be obtained from a slump test. The addition of water to increase slump will not compensate for a deficiency in fines, and coarse harsh mixes are not easily pumped. An optimum quantity of sand is therefore more important from a pumpability aspect than is any variation in consistency within the workable slump range of 2½ in. through 6 in.

Nor is the mere addition of sand sufficient. The sand as well as the coarse aggregates must be properly graded so that the mix hangs together and when a slump cone is stripped and tapped with a rod it will not fall apart but will remain in a plastic mass exhibiting no gritty or gravelly appearance. The cumulative grading curve of the combined aggregates and sand should be fairly smooth with no steep sections representing excess of certain sizes and no extensive flat sections representing lack of other sizes. Fortunately the grading curve for the Enders aggregates was relatively smooth. Only occasionally were excessive amounts of the 30 to 50 mesh size and shortages in the 8 to 4 mesh size encountered. Although the sand from river deposits was slightly angular and weathered, it had an average fineness modulus of 2.81. Most of the cement was relatively fine having a specific surface of about 3400 sq cm per g as determined by the Blaine air-permeability method. All aggregates were stockpiled in sidehill bunker bins over a conveyor belt and occasional segregation in the two coarse aggregates at the bunkers was unavoidable. When whole batches containing only the coarsest of either the 1½-in. or ¾-in. material reached the pump, plugging trouble was imminent. A shortage of the 1½-in. natural aggregate developed during the summer work and the supply of this size material was supplemented by crushed rock shipped from an adjoining state. The crushed rock, although mixed with the rounded river gravel, appeared to in-

crease slightly the resistance of the concrete to pumping.

Enders Dam is located in an area where damage to concrete by alkali reactive aggregate has been prevalent and, although a low alkali cement was used, it was considered advisable to take all the precautions possible to produce a superior concrete. Only the minimum amount of sand commensurate with good workability was used since this required less water and consequently did not impair the quality of the concrete. Test results on the air-entraining concrete after 28 days of water curing showed an average compressive strength of 4140 psi and a durability resistance of over 1200 cycles of alternate freezing and thawing.

SUMMARY

Since the introduction of the pumped concrete method in 1932, the use of concrete pumping equipment has been steadily expanded and progressive improvements have been developed and added. Undoubtedly this method of placing concrete will continue to advance and be viewed with more and more favor as both the technique and equipment are improved. It is understood that experiments with lightweight metal in pipeline fabrication have been undertaken and that some work has been done with rubber tubing as a means of increasing the flexibility at the discharge end. In all probability the pumps will be improved so that greater pressures may be developed and the possibility of pipeline plugging will be reduced. At present the flow of concrete in the line comes to a complete stop, or nearly so, after each pressure stroke and much energy is absorbed in accelerating the mass to its peak velocity again. The replacement of pulsating flow with continuous movement would do much toward maintaining uninterrupted work. The project manager for the contractor on Enders Dam expressed satisfaction with the use of pumped concrete on the spillway work and indicated that a substantial saving resulted on the concrete placements in areas not accessible by crane and bucket equipment. The capital investment in a crane and bucket versus that required for pumping equipment also offered some inducement to use the pumping method.

ORGANIZATION

L. N. McClellan is the Bureau's Chief Engineer and U. V. Engstrom is the Project Construction Engineer. The Kansas River District in which the dam is located is managed by H. E. Robinson, and is a part of Region 7 directed by A. A. Batson.

The Wunderlich Construction Co. of Jefferson City, Mo., in a joint venture with the James and Phelps Construction Co. of Oklahoma City is performing the work. Guy H. James is Project Manager.

In September **Letters from Readers** replaced **Job Problems and Practice,** with broader scope, more flexible form, and we hope more adaptability to the thoughtful, busy reader's use, as a forum for opinions, experiences and inquiries within the field of ACI activity. A letter does not need the studied treatment of a major paper to present valuable ideas for ready access to kindred minds.

Letters from Readers*

BY WAY OF SYNOPSIS

E. Y. Bragger adds further to the Editor's answer in the March Journal on the use of glass and clays in colored concrete.

Werner K. Gengelbach seeks information on lightweight concrete, especially gas concrete. The Editor gives a partial answer—more comments concerning gas concrete are invited.

Myle J. Holley, Jr., questions present methods of teaching reinforced concrete design.

Anton A. Anjel asks for information on the difference between air entrained in ordinary concrete and that resulting from the addition of an air-entraining agent.

Ceramic and Glass Aggregates for Colored Concrete (LR 45-15)†

E. Y. Bragger, Shawomet, R. I., provides further information on the use of clays and glass in colored concrete

Having visited the United Kingdom frequently from 1919 to 1939, I observed that there are few, if any, colored aggregates suitable for colored concrete, but the quarries of France, Belgium and Italy offer an abundant source of colored stone.

The Cement Marketing Co., Ltd., conducted tests at their laboratory in London and did some work at Brighton and other locations on the

*A part of copyrighted Journal of the American Concrete Institute, V. 20, No. 10, June 1949, *Proceedings*, V. 45. Separate prints are available at 35 cents each. Address 7400 Second Boulevard, Detroit 2, Mich.

†See also ACI Journal, Mar. 1949, p. 553.

channel coast. Large pieces of glass obtained from colored bottles were used. As far as the writer knows, no effort was made to crush and grade the glass aggregates.*

Vitrified Staffordshire clays, used in the famous Staffordshire bricks, will burn to a glossy jet black. The culls from these brickyards would probably make ideal aggregates when crushed and graded.

Some years ago, the Penn Tile Works Co., Inc., Aspers, Pa., produced fine and coarse colored aggregates in blue, light green, black, brown, yellow, pink, buff and white from clays obtained in the vicinity.

In conclusion, crushed and properly graded portland stone, when mixed with equal parts of gray and white portland cement (preferably waterproofed white portland cement), will produce concrete of a pleasing color with lasting and nonabsorbent qualities.

Gas Concrete and Perlite for Lightweight Concrete (LR 45-20)

WERNER K. GENGELBACH, Alamogordo, N. M., asks for information on lightweight concrete

I am interested in the use of lightweight concrete for the construction of small homes. I know that gas concrete was successfully used in Germany using a chemical manufactured by the I. G. Farbenindustries under the trade name "Iporit."

Can you tell me where gas concrete has been used in the United States and what building codes say about such construction? Are there test data on the strength of gas concrete? Where and under what trade name can suitable chemicals be obtained?

Referring again to lightweight construction, information is also desired on perlite.

As yet, to our knowledge, gas concrete has not been widely used in the United States although lightweight-aggregate concrete has been successfully employed in this country for more than 50 years.

Novel developments in lightweight concretes are described in an OTS† report on German building materials by the British Intelligence Objectives Sub-Committee.

Lightweight foamed concrete made by the hydrogen peroxide process was considered one of the most interesting developments in German

*Later work with glass aggregates indicates that they should be used with great caution as they may cause abnormal expansion when used as aggregate in concrete or mortar that is exposed to the weather.

†Office of Technical Services, Dept. of Commerce, Washington 25, D. C.

building materials. This process was patented by Deutsche Gold-und-Silberscheid Anstalt which also sponsored research on foamed plaster and nailable foamed concrete made with paper pulp. The hydrogen peroxide process was said to be simpler and easier to control than the aluminum powder process for making foamed concrete. The bubbles formed by liberated gases were more nearly uniform in size and the process could be used with any type of cement and under ordinary temperature conditions. The cost of the hydrogen peroxide process was slightly higher than the aluminum powder process. Other German foamed concretes were made with "Iporit," an I. G. Farbenindustrie product employing a sulfonic acid wetting agent, and the Asmus process which produced a foam with 10 percent aluminum powder and 90 percent calcium carbide.

Carl A. Menzel conducted tests on the strength of concrete as affected by admixtures of powdered aluminum. The following paragraphs are abstracted from Mr. Menzel's paper* concerning the chemical reactions between small particles of aluminum and moist cement paste and their effect on concrete.

The reactions observed were of the same type that has been used with much larger quantities of aluminum powder in plastic cement-sand mixtures to produce the lightweight concrete known as "Aerocrete."

Admixtures of aluminum powder can be expected to affect in various degrees the homogeneity and the strength of hardened concrete as well as the bond resistance of embedded steel bars with the enveloping concrete.

It appears very improbable that a great improvement in compressive strength of concrete through the use of the gas-producing aluminum powder will be realized in either pavement, mass concrete, or in reinforced concrete. In fact the evidence indicates that the compressive strength of good plastic concrete will be lowered appreciably (about 15 to 20 percent) by the smallest amount (about 2.0 grams per 100 lb cement) of aluminum powder required to counteract settlement and improve the bond resistance of the upper bars in deep beams. More aluminum powder than this optimum amount causes a further sharp decrease in strength and will also reduce the bond resistance.

It was noted that when the expansion of plastic concrete containing aluminum powder is purposely restrained, the concrete is subject to internal pressure during the early hardening period which tends to increase its compressive strength to values higher than that of similar concrete without aluminum powder. This is only further evidence that the strength of concrete both with and without aluminum powder is increased by pressure.

Tests made in this and other laboratories have shown that under proper conditions of use an admixture of aluminum powder may be very beneficial in improving the bond resistance of horizontal reinforcing bars in deep beams, in improving the effectiveness of grout in filling joints, bedding of heavy objects, or in improving the homogeneity of grouted concrete.

ACI Committee 212 reported that aluminum powder when added to mortar or concrete reacts with the hydroxides present in hydrating

*Menzel, Carl A., "Some Factors Influencing the Strength of Concrete Containing Admixtures of Powdered Aluminum," ACI JOURNAL, Jan. 1943, *Proc.* V. 39, p. 165.

cement and forms minute bubbles of hydrogen gas throughout the cement-water matrix. The amounts of powder added are usually small except in the production of the lightweight concrete of low strength.

The effect on strength depends to a considerable extent on the degree to which the tendency of the concrete to expand is restrained; without restraint the loss in strength may be considerable, but with a proper degree of restraint imposed the strength may not be affected appreciably and in some cases may be slightly increased.*†

Although perlite is not yet widely used in the construction field, considerable information is available on it and other lightweight aggregates.

Perlite is one of the natural volcanic glasses and is related in origin to pumice. To understand how perlite, obsidian, and related materials are formed, it is necessary to visualize the action of a volcano. At the time of an eruption, many fissures are created which radiate from the central core of the volcano and penetrate the rock strata in the surrounding area. These fissures are filled with erupting lava, which cools into various forms of volcanic glass, including perlite, obsidian, pumice, pitchstone and glassy tuff. The chemical character and physical appearance of these glasses depends on the composition of the parent lava, the type of eruption, the extent of flow through surrounding mineral bodies and admixture by them, changes of viscosity owing to its own content of water as well as outside contact with water, and rate of cooling and solidification.

The most common species of perlite was so named because it is generally a soft pearl gray or grayish black, closely resembling myriads of tiny pearls compacted and bound together. In fissures at some distance from the center of eruption the pearly globes are slightly larger than the head of a common pin; nearer the volcanic core they may be half an inch in diameter. Still nearer the core these shapes give way to cubes and prisms, then to long closely pressed splinters, then to massive material with distinct shrinkage fractures containing small inclusions of gas.

When processed, each of these kinds of perlite produces a different expanded material. Some types expand and bleach to a pure white by a swelling rather than an explosive action, and contain innumerable tiny bubbles, each with an extremely thin outer wall. Such particles may be very soft, powder easily, and have little compressive strength. After expansion the material has an average weight of 8 to 16 lb per cu ft, and is useful as an aggregate for lightweight blocks and slabs, for thermal and sound insulation. Further expansion produces fines weighing as little as 4 lb per cu ft and useful principally for insulation.‡

Usually, expanded perlite is produced only in the sand sizes. Concrete made with expanded perlite has a unit weight ranging from 30 to 65 lb per cu ft and strengths ranging from 100 to 1600 psi. . . . Some of the trade names under which expanded perlite is sold are: Permalite, Superlite, Ryolex, CBM Aggregate, and PerAlex.§¶

*"Admixtures for Concrete," ACI JOURNAL, Nov. 1944, *Proc.* V. 41, p. 73.

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"Porous Cement Products," *Concrete*, May 1929, V. 32, pp. 108-9.

‡"Lightweight Aggregate for Concrete," Office of the Housing Expediter, Washington, D. C.

§Price, Walther H. and Cordon, William A., "Tests of Lightweight-Aggregate Concrete Designed for Monolithic Construction," ACI JOURNAL, Apr. 1949, *Proc.* V. 45, p. 581.

¶Further information can be found in the following references:
Kluge, Ralph W., Sparks, Morris M. and Tuma, Edward C., "Lightweight-Aggregate Concrete," ACI JOURNAL, May 1949, *Proc.* V. 45, p. 625.
Ralston, Oliver C., "Perlite, Source of Synthetic Pumice," *Information Circular* 7346, Bureau of Mines, U.S. Dept. of Interior.
"Properties of Concrete Made with Typical Lightweight Aggregates—Housing and Home Finance Agency Research Program," *Material Laboratories Report* No. C-385, June 8, 1948, U. S. Dept. of Interior, Bureau of Reclamation, Branch of Design and Construction, Denver, Colo.

The use of lightweight concrete in building construction has been stimulated in recent years by the scarcity of other building materials, the saving in over-all construction costs which sometimes results from its light weight, and the experience which has been gained in its use.

—Editor

Teaching Reinforced Concrete Design—What Should We Emphasize? (LR 45-21)

PROF. MYLE J. HOLLEY, JR., Department of Structural Engineering, Mass. Institute Technology, Cambridge, Mass., discusses teaching methods

Certainly no one (least of all the teachers) would suggest that the courses offered by our engineering schools cannot be improved. Significant changes in course content or teaching method should reflect the experiences and requirements of men active in the profession—both those who design and those who employ designers.

With the hope that it may stimulate further contributions by others, the following brief discussion is offered.

At the undergraduate level, the objectives of a typical course in reinforced concrete design are (1) to familiarize the student with currently accepted theory for the elementary cases of flexure and direct stress; (2) to introduce the common structural forms to which reinforced concrete is particularly applicable; and (3) to further the student's training in design.

To achieve the first objective considerable laboratory drill is required. The elementary, linear theory certainly does not tax the student's mental ability, but the desired facility in its application necessitates the solution of numerous practice problems. The writer believes that instruction in the use of tables and charts should be postponed until the student has learned to solve problems without such aids.

The second objective can be achieved by student design of various types of slabs, beams, columns, footings, etc. While student familiarity with the simple beams can be attained, the average undergraduate course is too brief to permit but an introduction to the more complex structural elements. In the matter of floor slabs, for example, the instructor may properly hope that his students will learn: (1) the names and distinguishing features of the common types, (2) the conditions for which certain types are particularly appropriate, (3) a knowledge of some of the factors which make slab analysis so much more difficult

than the analysis of slender prismatic elements, and (4) a knowledge that tabulated coefficients are available to facilitate the design of slabs. The student should design one or more of each of the common slab types, but quite obviously time will not permit enough such design problems to develop any considerable familiarity.

The third objective, furthering of the student's training in design, would have varying significance for different instructors. The usual course in reinforced concrete teaches design in the restricted sense of adding to the student's tools of design, familiarizing him with typical specifications, etc. The extent to which the student's judgment (or feeling for structural form) is advanced depends largely upon the instructor. It is probably true, however, that in the average undergraduate course very little time is allotted to design in its broad aspects—despite the fact that most instructors are agreed on its importance. The attempt to inform the student on specific structural forms and the accepted methods for their design is time-consuming.

The objectives stated, the course content and methods indicated for their achievement are believed to be typical. The over-all aim of such a course is to maximize the student's initial value to an employer. An improved course might be designed to increase the student's knowledge of structural action of reinforced concrete, at a penalty, perhaps, of somewhat reduced initial design skill. Such a course might be based on a critical review of the research which has led to our present design formulas and specifications. The major subdivisions might be columns, beams, and slabs. In each case the students would trace the pertinent experimental research and its reflection in the successive changes in the codes. Certainly there would be no time for drill in the particular theory currently popular; no time for familiarization with charts and tables; no memorizing of requirements for "ordinary anchorage," or other code details. Might not the student exposed to such a course have a less superficial understanding of reinforced concrete than the student who is exposed to the more conventional design course? Would he not quickly overcome his initial disadvantage, and be of greater value to his employer through his more thorough background?

The foregoing comments leave much unsaid: the need for correlation of laboratory exercises with design courses; the relative importance of training in structural drafting as related to concrete design; the proper division of subject matter between undergraduate and graduate courses; the desirability of instruction in such matters as the plastic theory, prestressed concrete, etc. The writer hopes, however, that comment will be forthcoming from others on these many phases of teaching reinforced concrete design.

What Is the Difference Between Air Entrained in Ordinary Concrete and That Resulting from an Air-Entraining Agent? (LR 45-22)

ANTON A. ANJEL, Balboa, Canal Zone, poses a question

After reading the articles dealing with air-entraining concrete published in the February JOURNAL, a question arose concerning air entrainment.

I had assumed that concrete, with no air-entraining agent added, usually had somewhere around 1 or 2 percent of air entrained in relatively large bubbles, and that this air was not beneficial to the concrete. Messrs. Scripture and Litwinowicz have measured air contents which varied between 0.6 and 3.2 percent in concrete with no air-entraining agent added.*

The general consensus is that for best results, from 3 to 6 percent of air should be entrained by the air-entraining agent. Messrs. Blanks and Cordon state that "tests have revealed that when the air content of a concrete containing 1½-in. maximum aggregate drops below 2 percent, the resistance to freezing and thawing decreases appreciably."† This 2 percent minimum is less than the amount of air entrained in the concrete with no air-entraining agent added in several of the tests made by Messrs. Scripture and Litwinowicz.

Just what is the relation between the air (which I presume is in larger bubbles) entrained in the concrete with no air-entraining agent added, and the air (in minute bubbles) entrained by the agent? Are the larger bubbles entrained along with the smaller ones when the air-entraining agent is added to concrete, or does the action of the agent reduce the size or quantity of the larger bubbles? If the same quantity of the larger bubbles is entrained with or without the addition of the air-entraining agent, then a concrete with 3 percent of measured air might not contain more than a fraction of 1 percent of the kind of air that gives it the properties of air-entraining concrete.

*Scripture, E. W., Jr. and Litwinowicz, F. J, "Some Factors Affecting Air Entrainment," ACI JOURNAL, Feb. 1949, *Proc.* V. 45, p. 433.

†Blanks, R. F. and Cordon, W. A., "Practices, Experiences and Tests with Air-Entraining Agents in Making Durable Concrete," ACI JOURNAL, Feb. 1949, *Proc.* V. 45, p. 469.

Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

Concrete floors on ground

Concrete Information, No. CP 25, Portland Cement Assn., Chicago, 4 pp.

Proper subgrade preparation and granular fill to provide necessary support and drainage for slabs on ground are described. Provisions for utilities, damp-proofing, concrete mix and placement, reinforcement, insulation and floor finishes are included.

Recommended practice for building watertight basements with concrete

Concrete Information, No. CP 24, Portland Cement Assn., Chicago, 4 pp.

General precautions are given to prevent settlement of footings and the accumulation of ground water along the outside of the basement walls. Typical designs and proper construction methods for concrete walls are described and illustrated. Finally, construction of the concrete floor is covered briefly.

Expressways

Bulletin No. 16, Highway Research Board

The bulletin consists of a Highway Research Board committee progress report and three papers on controlled access highways. The papers are concerned with parking and terminal facilities, the effect on city streets of stage construction of limited access highways, and expressway route selection and vehicular usage.

The use of electric resistance strain gages, and the effect of aggregate size on gauge length in connexion with the testing of concrete

R. D. Binns and H. S. Mygind, *Magazine of Concrete Research* (London), No. 1, Jan. 1949 pp. 35-39

The components and theory of electric strain gage circuits and the measurement of drift are discussed briefly. Experiments on the relationship of gage length to aggregate size are touched upon and the need for further research is indicated.

Airox concrete aggregate

S. G. Dolman, *California Journal of Mines and Geology*, V. 40, No. 1, 1944, pp. 131-33

Ceramic Abstracts
May 1949

A diatomaceous mudstone containing hydrocarbons is fired to make a light aggregate for concrete. The raw material is fed through two rotary kilns. The first kiln drives off

*A part of copyrighted Journal of the American Concrete Institute, V. 20, No. 10, June 1949, *Proceedings*, V. 45. Address 7400 Second Boulevard, Detroit 2, Mich.

the excess oil; the second kiln, maintained at a temperature of 2300 F, causes vesiculation of the feed. To prevent clinkering, silica powder is dusted over the pellet feed before it enters the second kiln. Each vesiculated pellet acquires a fused surface. Concrete made with this aggregate has 1.5 times the compressive strength of ordinary concrete but only 60 percent of the weight.

Building better farm homes with concrete

Portland Cement Assn., Chicago, 32 pp.

A number of styles of concrete homes with various architectural effects are shown after which construction details for concrete masonry and monolithic concrete dwellings are given. The requirements for good walls, insulation, concrete subfloors, footings, basements, stairways and roofs are considered. Suggestions for modernization, planning a new home and concrete improvements conclude the phamphlet.

The strength of reinforced concrete beams in shear

A. Sunderland, *Magazine of Concrete Research* (London), No. 1, Jan. 1949, pp. 3-8

The results of tests of 39 beams designed to fail in shear or diagonal tension are reported. The factors investigated were spacing of vertical stirrups and percentage web reinforcement and the ratio of depth of beam to span.

Observed shearing failing loads are compared with loads derived from British and American Codes of Practice with the conclusion that the British Code is too conservative.

Electric curing of concrete

J. D. McIntosh, *Magazine of Concrete Research* (London), No. 1, Jan. 1949, pp. 21-28

Concrete used to re-lay small areas in cold storage rooms was heated by a constant voltage alternating electric current passing through it. The relationship between compressive strength of this concrete and the same mix cured normally was found to vary with the maximum temperature and time of treatment. The possibility of using electric curing for accelerated compressive strength tests is discussed briefly and the application of the method to the repair of floors in cold storage rooms is outlined.

Bricklaying

Delmar Publishers, Inc., Albany, N. Y. 203 pp. $2.75

This manual for instruction in the laying of bricks and concrete masonry units was developed for use in the building trades and secondary and post secondary schools.

In addition to manipulative skills each unit of the book includes related information, tools, equipment and safety, trade mathematics and estimating, trade drawing and details of construction and shop practice. The 14 units represent a suggested first year course for the beginning apprentice bricklayer.

Measurement of the thickness of concrete pavements by dynamic methods: a survey of the difficulties

R. Jones, *Magazine of Concrete Research* (London), No. 1, Jan. 1949, pp. 31-34

The practical difficulties and errors in methods used to determine the thickness of concrete pavements by the propogation of sound waves in the slab are reviewed. Calculations are given which show that the method based on the reflection of an ultrasonic pulse by the bottom of the slab would give accurate results if present experimental difficulties are overcome.

A simplified method for computing the bearing capacity of the soil supporting footings or piers

L. Marivoet, *Proceedings*, Second International Conference on Soil Mechanics and Foundation Engineering, 1948, V. 1, pp. 78-86 Applied Mechanics Reviews Mar. 1949 (Hill)

Graphs are presented of the bearing pressure of continuous footings as a function of the depth and breadth of the footing, and of the angle of internal friction. The calculations are based on an empirical extension of the Prandtl formula for a cohesionless material. Graphs are also given for the settlements which are calculated by Terzaghi's method of influence coefficients.

Raising of concrete slabs

F. Schutz, *Betong* (Stockholm), V. 33, No. 4, 1948, pp. 159-165 Reviewed by Eivind Hognestad

The present paper gives a brief description of the mud jack method as applied in the United States for raising concrete pavement slabs. The materials, procedures and equipment for filling voids under pavement slabs are outlined, and particle size curves are given.

This method has been used with success in Sweden for the raising of floor slabs as well as highway pavements.

Concrete masonry

Amund M. Korsmo, *Technical Bulletin*, Housing and Home Finance Agency, Washington, D. C., No. 9, Mar. 1949, pp. 7-17

Types of concrete masonry units are described and modular dimensioning is mentioned. Current standards of quality are listed as are leading manufacturers of concrete block machinery.

Properties of aggregates, mix proportions, and curing of concrete units are covered briefly. Acoustical properties, insulating value, fire resistance, rain penetration, and exterior treatment are touched upon.

Raising of a concrete floor by a grouting method

P. Hallström, *Betong* (Stockholm), V. 33, No. 4, 1948, pp. 166-170 Reviewed by Eivind Hognestad

A concrete floor, about 65,000 sq ft, was raised by means of grouting during 1945-46. The floor in an assembly hall of an automobile factory was poured on a soft clay soil. The superstructure was built on a pile foundation. Differential settlements of the floor as large as 12 in. took place. The floor was raised very satisfactorily with a remaining maximum deflection of ½ in. by means of grouting with a mixture of cement, clay, fine sand and water. The water-cement ratio was about 3.10, and the cube strength after 9 weeks was 260 psi.

Nationally recognized standards in state laws and local ordinances

Report by Committee Z 56, American Standards Assn. 44 pp. $1.00

A study of methods which might provide more effective use of nationally recognized standards as a basis for state and local regulations, this report includes two papers emphasizing the need for uniformity in state and local regulations and the effects on business and the public of the present lack of uniformity in legal requirements. A third paper summarizes the present status of the "adoption by reference" method of utilizing national standards and furnishes the texts of enabling acts adopted by 16 states. The fourth paper discusses the legality of several methods that have been followed to provide for the use of national codes and standards as a basis for local regulations.

Foundation work for a warehouse, Port of Dublin

B. A. NOLAN, *Bulletin* No. 6, The Institution of Civil Engineers of Ireland, Trans. V. 75, Apr. 1949, pp. 145-157

Settlement of buildings adjacent to the proposed site of a 4-story, 470-ft. long warehouse in Dublin led to an exploration by boreholes and test piles. The test piles used were 4½-in. diameter steel tubes coupled in 10-ft. lengths and shod with a 6½ in. diameter cast-iron shoe which remained in the ground when the tube was withdrawn. Loading data obtained were related to the permanent piles by the Hiley formula. The substrata conditions found and conclusions reached are given in some detail. The design, construction and test loading of the cast-in-place concrete piles used are discussed briefly.

The Wigmore consistometer

Roads and Road Construction (England), V. XXVII, No. 315, Mar. 1949

HIGHWAY RESEARCH ABSTRACTS Apr. 1949

The Wigmore consistometer, manufactured and used in England, measures by penetration and vibration the workability of concrete and records a consistency factor covering a wide range of mixes. The apparatus consists of a portable container through which a bronze ball mounted on a steel rod moves vertically. The rod is centralized by passing through the cover which fits over the rim of the container.

The operation of the consistometer, which is given in detail, can be performed by one man in less time than required for a slump test.

Behavior of plaster and its supporting structure under various loading conditions

JACOB D. DUMELLE, DALE D. DOERR and H. ROBERT NELSON, the *Frontier*, Armour Research Foundation, Mar. 1949, p. 3.

In an experimental program on standardized lathing and plastering practices and their effect on the behavior of plaster construction, tests were made of the physical properties of various plaster mixes, the effect of plaster thickness, lath placement and reinforcement at window corners on the crack resistance of plaster wall construction, and various ceiling construction techniques.

The effects of plaster-sand ratios and thickness on plaster coats, size and type of lathing material and method of fastening are discussed and the need for research into the use of lightweight aggregates is indicated.

Beam factors and moment coefficients for members with intermediate expansion hinges

Modern Developments in Reinforced Concrete, No. 23, Portland Cement Assn., Chicago, 23 pp.

Design data are given for reinforced concrete members with expansion hinges placed not at the supports but somewhere in the span. They are used to provide for volume changes in long structures such as elevated expressways, viaducts or stream crossings.

Carry-over and stiffness factors as well as coefficients for fixed end moments in members with expansion hinges are presented in a manner suitable for use with the moment distribution method of frame analysis. General formulas are given for symmetrical and unsymmetrical haunched members, and tables are presented for members with soffits consisting of a full symmetrical parabola extending from support to support. Moment coefficients for symmetrical members with parabolic soffits are also reproduced as curves or influence lines.

The use of portland-pozzolan cement by the Bureau of Reclamation

ROBERT F. BLANKS, Paper presented at Portland Cement Assn. meeting, May 11, 1948. 44 pp.

Trends in cement specifications and historical uses of pozzolanic materials are mentioned briefly as a prelude to giving previous use of pozzolans by the Bureau of Reclamation in such structures as Friant and Altus Dams.

Heat of hydration and volume change, tensile and compressive strengths, sulfate resistance, extensibility, leaching, permeability and durability of portland-pozzolan cement concretes are examined and found, in the main, to recommend its use in mass concrete. The use of fly-ash as a pozzolan and its effect on the above properties is then given.

It is concluded that appropriate combinations of portland cement and suitable pozzolanic materials impart to concrete many of the qualities sought. At the same time, it is pointed out that there still are many unknown or incompletely understood factors affecting the action of pozzolans.

Studies in cement-aggregate reaction, Part VIII—The expansions of composite mortar bars

H. E. VIVIAN, *Journal of the Council for Scientific and Industrial Research*, V. a1, No. 2, May, 1948, pp. 148-159

Reviewed by D. L. BLOEM

The author describes tests in which composite bars composed of varying numbers of alternate sections of mortar made with high or low alkali cement and reactive or non-reactive aggregate were tested for expansion due to alkali-aggregate reaction. It was found that expansion increased as the number of sections was increased in bars containing both high alkali cement and reactive aggregate regardless of whether or not the reactive ingredients were in direct contact in the original mortar. There was evidence of alkali migration from sections containing high alkali cement to sections containing low alkali cement and subsequent attack on the reactive aggregate in the latter sections. The author attributes the increase in expansion with increased number of sections largely to the greater availability of water from areas adjacent to the reactive ingredients.

Prestressed concrete railroad ties

O. D. LAERUM, *Teknisk Ukeblad* (Oslo), V. 96, No. 7, 1949, pp. 97-101

Reviewed by EIVIND HOGNESTAD

The precast concrete tie is not a new idea. Italian railroads started experiments over 40 years ago, and the concrete tie has also received considerable attention in the U.S.A. Mainly due to manufacturing cost, however, the wooden tie still is most used.

English industry has, in the last decade, reached satisfactory results with prestressed concrete ties. The author, a professor of railroad engineering, visited the English factory at Tallington in June 1948. In this article he gives an outline of the manufacture. Practical experience with these ties in the field has been encouraging in England as well as in Egypt, Palestine, South Africa and India. The main disadvantage of the concrete tie is that it weighs 150 percent more than an impregnated wooden tie. The price, however, is now nearly the same in England, and it can be expected in the near future that the scarcity of wood and new applications of this material will change the prices still more in favor of the prestressed reinforced concrete tie.

Dimensioning of reinforced concrete conduits

G. WASTLUND and S. EGGWERTZ, *Betong* (Stockholm), V. 33, No. 4, 1948, pp. 171-216

Reviewed by EIVIND HOGNESTAD

This report deals with an investigation of the strength properties of underground conduits which was carried out in connection with a revision of the Swedish standard specifications for concrete culvert and sewer pipes.

In strength computations, the loads were assumed in accordance with the common theories of earth pressure as adapted and checked by tests for conduits in ditches and embankments by A. Marston at Iowa Engineering Experiment Station. In designing reinforced concrete pipes, the authors not only consider maximum stresses, but widths of cracks are also taken into consideration. A width of 0.2 mm at design loads for pipes with double reinforcement near the surfaces, and 0.5 mm for single reinforcement, were considered allowable without endangering the permanence of the structure.

Eighteen test specimens of internal diameter from 20 to 80 in. were manufactured and loaded to collapse to gain experimental experience for the high steel percentages considered in the new specifications. Loads at first crack and ultimate loads are reported as well as deformations and widths of cracks measured at certain intervals. Finally some design examples are given to facilitate the use of diagrams and formulas in design. Thirty-two references are listed.

Broad and narrow beam attenuation of 550-1400 KV X-rays in lead and concrete

H. O. Wyckoff, R. J. Kennedy and W. R. Bradford, *Nucleonics*, Nov. 1948, V. 3, No. 5, pp. 62-70

The energy conditions affecting barriers to X-ray beams are stated as a preliminary to describing an experimental arrangement for studying the efficacy of lead and concrete as shielding materials under narrow and broad beam conditions and at varying target distances. X-ray attenuation curves in lead and concrete are presented and discussed. The authors' conclusions pertaining to concrete are:

"For concrete protective barriers the difference between narrow and broad beam thickness requirements is of the order of 1.5 to 2 half-value layers depending upon the portion and kilovoltage of the curves considered. If the irradiated area of the barrier is not more than 3 ft in diameter and if all personnel are restricted to distances greater than 10 ft from the barrier then narrow beam attenuation curves may be used in designing X-ray protection. Such restrictions are not usually feasible, however, because of space limitations. In addition, the cost of requiring the extra space may be larger than the saving in barrier cost.

For the majority of X-ray installation designs, therefore, the broad beam attenuation curves should be used."

Concrete-spliced wood piles answer need for 55-ft lengths in Venezuelan dock

Robert G. Dykton, *Civil Engineering*, Jan. 1949, p. 36-37 Reviewed by J. R. Shank

Fifty-five-ft piles were needed for breasting and mooring dolphins at a dock in Lake Maracaibo, Venezuela where it was difficult to find timbers long enough. Splicing timber piles of vera or imported southern pine was decided upon. The 6-ft splices were made of reinforced gunite. Reinforcing consisted of 12 ¾-in. longitudinal bars inside a cage of No. 10, 4 x 6-in. wire mesh with five spiral turns of ¼-in. wire at top, bottom, and center. The longitudinal bars were placed so that they were covered on the outside with 2 in. of gunite. The gunite was 5 in. thick all around. At least 4 in. of gunite was shot in between the ends of the piles. The gunite was of 3:1 proportion using bank-run sand.

Piles were tested at the age of 14 days, first by picking them up 10 ft from one end and then by driving and bending after being driven. One pile of vera in a 4-in. sleeve, which was very crooked, shattered during driving under a drop energy of 35,000 ft-lb. A second pile of the same type and size, though straight, was driven with a maximum drop energy of 40,000 ft-lb. It showed no signs of structural damage. When a bending moment of 300,000 ft-lb was applied, the longitudinal bars pulled out. A southern

pine pile with a 5-in. sleeve withstood the same sort of test and broke off completely at the mud line, below the splice.

The larger splices were estimated to cost $33 each.

The effect on concrete of exposure to sea water

C. R. Bickford, *Journal of the Institution of Engineers* (Australia), Dec. 1948, V. 20, No. 12
Reviewed by J. R. Shank

Though the author uses the word "Concrete" in his title, he limits his paper to reinforced concrete and assumes penetration of the salt water to the steel. He discusses the electrolytic action of corrosion of reinforcing in concrete rather extensively. He makes these statements: "The difficulties that have arisen in the past with regard to concrete being affected by sea water have been due largely to defective aggregates or, in addition, in some instances, to the poor quality of the concrete with insufficient and unstable cementing material." "It seems to be a generally accepted fact that if the concrete is of such a quality as would prevent the salt water obtaining access to the steel reinforcement, no deterioration of the concrete need be expected, as it is the pressure of the salt that constitutes the agent contributing to the corrosion of the steel and subsequent breaking down of the concrete cover."

The author sheds no light on any agents acting to deteriorate the concrete cover. He recommends bituminous or other waterproofing membranes for surfacing concretes to work in sea water. He cites a good method of bonding such membranes to the underside of beams, slabs, and girders, mentioning that such surfaces even though not touched by the water are susceptible to deterioration. Impervious concrete with sufficient cover over the steel is essential.

Notwithstanding the generalities cited he mentions that deterioration occurs largely above low water mark and not in that portion continuously submerged.

Surface available to nitrogen in hydrated portland cements

R. L. Blaine amd H. J. Valis, *Journal of Research*, National Bureau of Standards, V. 42, No. 3, Mar. 1949
Reviewed by C. C. Fishburn

Measurements were made of the adsorption of nitrogen, oxygen and argon on samples of hydrated portland cement. The effect of granulating the specimens and of the temperature at which specimens were evacuated, as well as the effect of different water-cement ratios used in preparing the neat paste, of the temperature and conditions of storage and the effect of age on the apparent surface of test specimens were studied.

Surface area values calculated on the basis of the Brunnauer-Emmett-Teller theory indicated an increase in surface available to nitrogen with age of test specimens when a high water-cement ratio was used in preparing the paste but not when a low water-cement ratio was used. The temperature at which a specimen was evacuated greatly affected the surface available to the nitrogen molecules. Granulating the specimens resulted in greater surface area than was obtained with the whole specimens but the apparent surface areas decreased with decrease in size of the granules. Temperature of storage and type of moist storage up to 28 days made little difference in the apparent surface available to nitrogen. After 28 days the water stored specimens had greater apparent surface area than did the moist air stored specimens.

Most of the tests were made with a Type I cement; a few tests were made on Type II and Type III cements. Different surface area values were obtained with the different cements. Surface area values calculated from the N_2, O_2 and A isotherms were of the same order of magnitude. They were, however, much smaller than those previously reported, which were calculated from the adsorption of water vapor.

Thin concrete arch roof provides 340 ft clear span for bomber hangar

Louis W. Prentiss, *Civil Engineering*, Feb. 1949, pp. 34-38 Reviewed by J. R. Shank

A 314½ x 369-ft bomber hangar at Rapid City, S. D., has 25-ft span Z-D construction 5 to 7-in. thick between 20 x 60-in. arch ribs resting on relatively short heavy columns tied under the floor by six 2½-in. nickel-steel rods. The span of the arch ribs is 340 ft between columns. The footings are on concrete piles designed to take 44 tons each. Scale models of the roof, columns and footings were constructed for detailed studies of stresses.

A system of adjustable and movable centering and roof forms costing more than $100,000 was designed and details of construction procedure were worked out in advance. Two bays, 50 ft wide, were constructed at one placing of the centering. Pressed and matched sheathing laid diagonally for the roof forms was lined with masonite. Concrete was handled to hoppers by steel towers 120 ft high, and placed by buggies and short chutes. Air-entraining concrete containing 4 percent air, having slumps of 3 to 5 in. and strength of 3750 psi was used. Concrete for roof slabs was finished with hand floats and push brooms on the steeper slopes and almost exclusively with bull floats on the flatter slopes. No screeds were used. Membrane curing compounds were used on exposed surfaces.

The architect-engineer's field representative computed the amount of vertical deflection anticipated at each of the 176 roof jacks for each stage in decentering. He also prepared a table of load increments to be applied in stressing the tension ties so that such stressing would always be one increment ahead of the loading of the columns. While stressing of the rods or lowering of the form jacks was in progress continuous readings were taken on permanent markers on each column to insure that no vertical, horizontal or rotational movements of the structure occurred. To free the forms, jacks had to be lowered until the observed vertical deflection at the crown was about $\frac{1}{16}$ in.

Erosion due to incipient cavitation

W. T. Bottomley, *Proceedings*, Institution of Mechanical Engineers (London), 1948, V. 158, No. 3, pp. 297-316 Applied Mechanics Review Feb. 1949 (Ghaswala)

The classic work on the complete life history of a cavitation bubble, comprising its growth, collapse and final rebound, was recently published by Knapp and Hollander ("Laboratory Investigation of the Mechanism of Cavitation," *Trans.* A.S.M.E., July 1948, pp. 419-435). In this paper, primarily based on the data supplied by van Iterson, the author advances certain ideas about the formation and collapse of bubbles causing cavitation erosion, and compares them with the existing theory of Cook and Parsons propounded in 1919 and by Rayleigh in 1920.

He suggests that the bubbles which cause erosion are ordinary air bubbles filled with air at roughly atmospheric pressure, and that their collapse is due to surface tension only and is not a result of the sudden change in water pressure. These suggestions are not beyond the range considered by others like Breeching and Numachi; nevertheless the discussion presented by the author portrays his own theory based on his experimental framework rather than a further development of the general theory of cavitation.

The experiments also show the vapor bubbles which collapse in deaerated water are in a state of thermal equilibrium. The energy which produces cavitation erosion is the free-surface energy liberated by the collapse of the air bubbles, while the final collapse velocity is the velocity of sound in water, with an impact pressure of 120 tons per sq in. or more. The actual effects of these ideas on the interpretation of model tests in cavitation tunnels are described.

There is an interesting discussion in which some of the author's ideas are refuted and shown to be invalid. This reviewer also feels that the true conception of the mechanism of cavitation erosion can only be had after these experimental results are fully evaluated. The paper, through very controversial in its nature, forms a lively springboard for further discussion and research, and a notable contribution to the widely scattered literature on cavitation erosion.

Testing of hardened concrete in structures without removing specimens (Zur Frage der zerstorungsfreien Prufung con Beton im fertigen Bauwerk)

A. KLEINLOGEL, *Betonsteinzeitung* (Wiesbaden), 1949, No. 2, pp. 17-21 Reviewed by RUDOLPH FISCHL

Since 1934 attempts have been made in Germany to work out a way of testing hardened concrete in structures by applying an impact to the surface.

Professor Gaede, Hannover, was the first to develop a spring-controlled hammer which causes a known impact on the concrete surface under test. The diameter of the impression allows the tester to determine the strength of the concrete. Using procedures similar to those of the Brinell hardness test, Gaede developed an empirical formula the results of which are in close agreement with those obtained by standard tests.

C. A. Einbeck, Coburg, recently developed much simpler tools based on the same principle. One is a hammer in the form of a pendulum connected to a holder. The head of the hammer contains a socket-mounted hardened steel ball 25 mm in diameter. The tester presses the holder by hand to the concrete surface, releases the hammer from full height, and catches it after one blow has been struck.

Diameters from at least 20 impressions are measured with a 1/10 mm scale and a magnifying glass. Average diameter of the impressions for a concrete of normal consistency ranges from 8 to 15 mm. The relation of the diameter of the impressions to the strength of concrete must be established for each type of concrete by closely correlated standard compression tests. This is important because of local variations in aggregates and consistency.

The second tool developed by Mr. Einbeck is a simple hand hammer with a 35 cm handle and the above mentioned steel ball. Many tests performed by the inventor and technical laboratories in Western Germany proved the definite relation of the diameter of the impressions to the strength of concrete. The objection that the physical strength of the operator governs the impact and thus influences the result is valid, but practical experience has shown that it is possible to acquire the ability to determine the strength of concrete by comparisons with standard test results.

The deviation of test results by the impact method is within reasonable limits, considering that even under excellent conditions results of standard tests may differ as much as -10 to $+43$ percent of design strength.

Tests by the impact method are not designed to eliminate normal control tests, but are intended to serve as valuable field supplements to them. Principal advantages of this method are its economy and the possibility of a continuous check of strength during construction without the usual waiting time.

Manual of Report Preparation

FRANK KEREKES and ROBLEY WINFREY, William C. Brown Co., Dubuque, Iowa, 1948, 397 pp. $6.00 Reviewed by MARY KRUMBOLTZ HURD

This manual of report writing has been prepared for both engineering students and practicing engineers who are called upon to prepare reports. The dual purpose may account for the great detail of presentation; each element involved in producing an

engineering report is described and categorized for the student, and then the preparation process outlined for the report writer. Samples of all parts of a report, including letters and questionnaires used in securing data, are presented in facsimile form, some in color.

Sections of the book will be of value to secretaries and typists who assemble and copy reports. There is much material on grammar, diction and style which may be of value to the general reader; authorities ranging from Descartes to the Standard Oil Company are quoted. The complete A.S.A. list of abbreviations for scientific and engineering terms is quoted, and Rudolph Flesch's formula for determining readability ratings is explained in full.

The extensive illustrative examples are an outstanding feature of the book; its letter-size pages facilitate reproduction at full size of most of the elements treated. If a work may be condemned for being too comprehensive, then that is perhaps the fault of this one. Much elementary material of necessity has been included for the student; the practicing engineer who seeks to use the volume for reference on an individual problem may, in spite of the table of contents, become lost in a maze of interesting (and time-consuming) material. An index would add materially to its reference value.

The first of four parts of the book is devoted to fundamental skills; including an analysis of the characteristics of good reports, sentence and paragraph structure, principles of outlining and importance of strong vocabulary. Possible sources of information for the report are classified, and techniques of exploiting each are presented. Principles of tabular organization of data are given, formulated with contrasting arrangements (good and bad) of the same data. A chapter on illustrations gives suggestions on drafting standards, mounting, and lettering planned for photographic reduction. Actual photostat and blueprint presentations of data and graphs printed on cross section paper are bound into the book.

The second portion devoted to "investigation, organization and content" treats planning and outlining the actual work of report preparation and presents samples of letter reports. The several elements of formal reports are discussed: title page, transmittal letter, foreword, preface, acknowledgment, contents, abstract, body, references, appendix and forms for citations. The body of the report is treated in terms of the introduction, opening sentence, choice of material, discussion, and solution of the problem.

A third section on manuscript preparation provides information on aesthetic considerations of the type-page layout, choice of heads and placing of illustrations. Mechanics of style, including abbreviations, spelling, capitalization and punctuation are presented in detail. One chapter is devoted to preparing and editing or revising the rough draft.

Supplementary sections deal with oral presentation of reports, and copyreading and proofreading for presentation of a report in print. The appendix includes two complete specimen reports, facsimilies of typewritten originals.

Symposium on prefabricated houses

Beton-Teknik (Copenhagen, Denmark), V. 14, No. 4, 1948, pp. 101-151
Cement och Betong (Malmö, Sweden), V. 23, No. 4, 1948, pp. 135-186

Reviewed by EIVIND HOGNESTAD

Prefabricated concrete houses

H. ROHFORS, Swedish Cement Assn.

Throughout the world the war intensified an already critical housing shortage. Lack of skilled labor, materials and coal for manufacture of bricks require the development of new methods of building homes. Concrete has several advantages. Local, or in any

case domestic, cheap materials can be used. Concrete also gives a strong, fireproof frame with low weight and relatively few skilled workers are needed.

Some systems of precast floors are mentioned, and some arguments are presented for the economy of precast concrete.

Small concrete houses
K. Hellberg, Swedish Cement Assn.

Due to the importance of saving wood for export, state as well as municipal governments and private firms have shown considerable interest in improving the construction of apartment houses. Lately, however, there has been an increasing demand for small one-family houses ordinarily made from wood. The Scandinavian Cement Assns. agreed in the fall of 1946 to investigate possibilities of building small houses of concrete. For individual homes various types of concrete blocks and lightweight concrete can be used. In Sweden, however, a system suitable for large scale, regional construction of small houses was considered desirable.

To obtain the necessary information regarding such houses it was decided to construct a test house using large wall elements cast horizontally at the building site. It was found that the concrete house could be built for the same cost as a prefabricated wooden house designed from the same architectural drawings. The cost of maintenance and fire insurance for the concrete house is expected to be less than for a similar wooden house. In addition, the concrete house makes possible a 60-70 percent saving of wood materials.

New building methods in Norway
St. W. Stephensen, Norwegian Cement Assn.

Two systems of prefabricated houses are described; one using large elements tilted up by a crane, the other using small, shop made elements easily handled by two men.

Christensen's and Nordnes's system, using large elements generally joined by welding of the reinforcement, has several interesting features. As compared to a normal concrete building, the consumption of steel and cement is 50 and 60 percent respectively. The use of wood for forms is very substantially reduced, as is the construction time which is reduced from about 2 months to 2 weeks for a two-story, four apartment building. The saving in cost of the building frame (walls, floors and roof) amounts to 20-30 percent.

Standal's system uses narrow wall elements one story high. The advantages are similar to the system mentioned above.

Types of prefabricated concrete houses in Denmark
E. V. Meyer

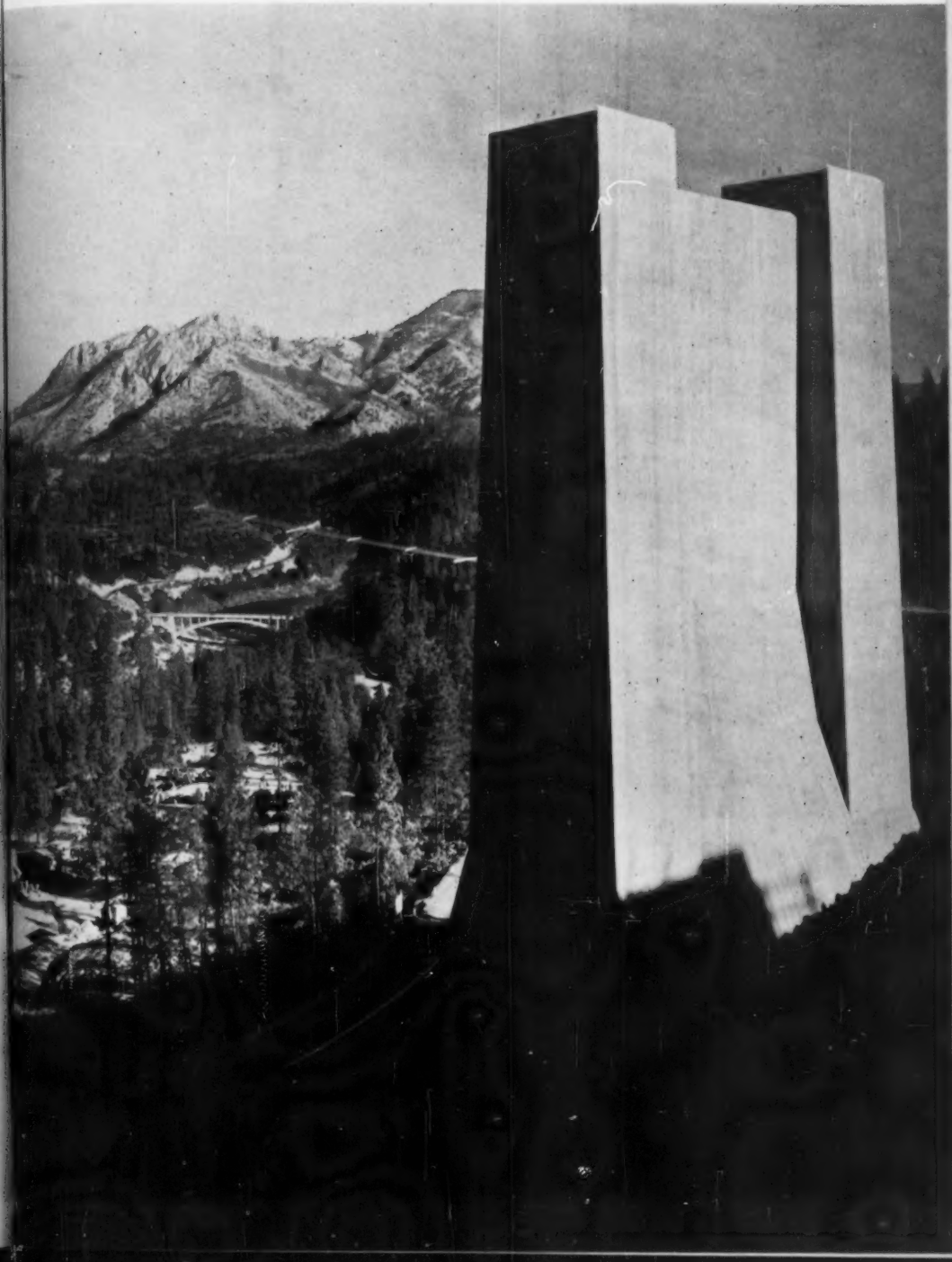
The increasing use of concrete by the housing industry in Denmark is caused, as everywhere else, by a shortage of wood and skilled labor. About ten different systems are described, the last three in some detail.

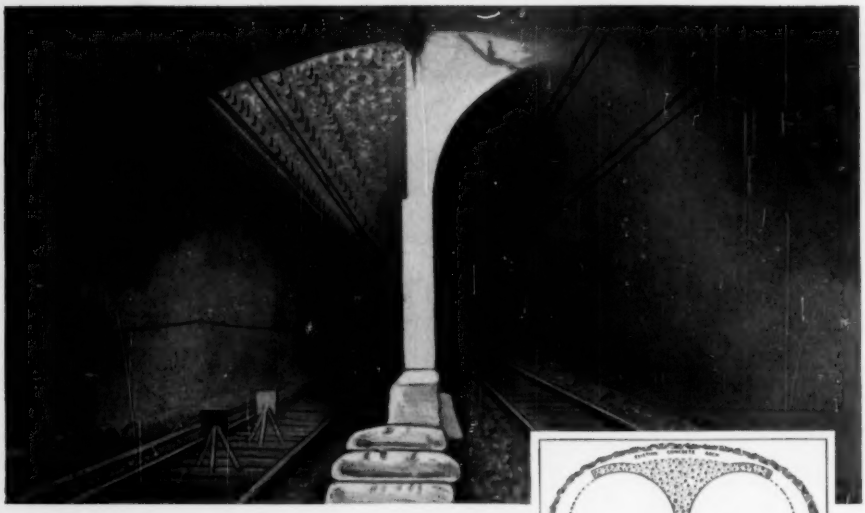
"Traebotex" elements are hollow blocks of an insulation material consisting of wood chips with cement paste as a binder. For wall construction these blocks (about 50 x 16 x 5 in.) are simply stacked on top of each other. The channels of each layer are filled with concrete.

Hafnia and MH houses use a precast concrete frame with insulation plates of the wood-cement mentioned above.

ACI News Letter

Vol. 20 No. 10 JOURNAL of the AMERICAN CONCRETE INSTITUTE June 1949





The above illustration shows two stages in the completion of the repair work. The old arch over the left track is prepared for the new arch construction. The arch over the right track has been completed. The section drawing shows the method of construction.

PREPAKT REPORTS ON . . .

FIRE REPAIR

Previous satisfactory results from the use of INTRUSION-PREPAKT methods and materials for rehabilitation of the original lining of the three-mile long Mount Royal Tunnel of the Canadian Pacific Railways led to the adoption of the same methods and materials for repairs to the tunnel after a disastrous 1946 fire.

The great damage to the tunnel necessitated extensive structural rebuilding without requiring a stoppage of rail traffic. Intrusion-Prepakt methods made it possible to carry out this extensive conversion from a single flat arched tunnel to the twin-arch structure shown above, without interruption to rail traffic. The difficult problem of the concrete placement was solved by routine Intrusion-Prepakt procedures, demonstrating the efficiency of using Intrusion-Prepakt in "hard-to-get-at places".

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ACI

News Letter Contents

June 1949

On the cover—Pier No. 1 of Pit River Bridge over Shasta reservoir. The completed bridge replaces the concrete arch bridge in the middle background now submerged by water impounded by Shasta Dam.

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New England regional meeting planned

Yielding with pleasure to repeated warm invitations from ACI Members in New England, the Board of Direction has scheduled a regional meeting to be held in Boston November 3 and 4 in connection with the annual fall Board meeting.

A local committee is being organized by ACI Directors Kennedy and Norton to lay plans for the meeting and to develop an interesting technical program of two or three session periods, a luncheon or dinner with a speaker of general interest, and opportunities for Members and non-Member visitors to mingle, exchange ideas and get better acquainted.

Details as to headquarters hotel, exact time of meetings, etc., are yet to be decided. More complete information should be available for publication in the September *News Letter*, and it is expected Members and prospective visitors in the adjoining region will receive more information by mail in late summer. While the event is planned as a regional meeting, all Members are cordially invited to attend. Members in and near New England will have opportunity to get together and bring those friends who might benefit by learning more about ACI.

San Francisco Selected for 1951 Convention

California now being ACI's top membership state, the Board of Direction made a bow to the West Coast; designated San Francisco for the February 1951 convention—provided satisfactory arrangements could be made.

ACI's Secretary-Treasurer explored the possibilities. He found a go-getting spirit among members in the Bay region who promised steam behind the event—arrangements for hotel and other accommodations are nearing completion. Put it on your calendar for the week of February 18, 1951. San Francisco is the place but all the Pacific coast and nearby states are to be called into action for ACI's first time west of the Rockies.

Are You Understood?

Frank Tatnall, ACI Member since 1930, Editor of Baldwin Locomotive Works' *Testing Topics*, sounds a call for intelligible technical writing and speaking which applies so aptly to the ACI JOURNAL—and to convention programs—that it is reprinted *in toto* with the author's permission.

Simple Language

Much comment followed the editorial in the last issue of TESTING TOPICS in which your editor sounded what apparently is a common complaint in the field of technical writing . . . the lack of "simple language." Technical writing, when simply done, has telling effect. To prove it we have only to cite a passage by a professional raconteur, Rudyard Kipling, describing a real-life experiment in the dynamic testing of structures.

" . . . The skipper said to the engineer, 'The parts of the new ship have not learned to work together yet.

'Every inch of her has to be livened up and made to work with its neighbor. Our new little ship has to be sweetened yet and nothing but a gale will do. All the hundreds of plates that are riveted to the frames echoed the call. Each plate wanted to shift and creep a little, and each plate, according to its position, complained against the rivets.'

"And later, after the gale, there was just as much groaning and straining as ever, but it was not so loud or squeaky in tone and when the ship quivered, she did not jar stiffly like a poker hit on the floor, but gave a supple little waggle, like a perfectly balanced golf club.

"For when a ship finds herself all the talking of the separate pieces ceases and melts into one voice which is the soul of the ship."

Kipling has painted a word picture that modern engineering writing couldn't produce in pages of elaborate words. Any laymen can understand Kipling.

At various times, your editor has served as Program Chairman for local sections of technical societies. During the course of normal association with engineering people, he uncovered in everyday conversation information which sounded so highly interesting that he signed up certain people immediately for formal talks. We fully expected that such stories as were uncovered in conversation would reach a new high in interest and presentation when given at the meetings.

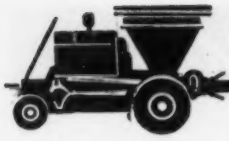
Then came the final presentation. The talks were hardly recognizable. They were no longer the homey, factual stories heard in conversation. They were stiff and formal . . . heavy, full of statistics. To accompany the talks, tables of figures were projected on the screen and no one could interpret them—even if they could read them—which is doubtful. Gone were the pointed conclusions—the terse summary of practical interpretation.

The speakers, apparently, tried desperately to protect themselves from the challenges of their colleagues, and, in doing so, lost all the interest and enthusiasm of the conversational presentations.

As each new page of printed technical literature passes this desk, your editor becomes a more enthusiastic exponent of the "simple language" presentation. He heartily disagrees with what seems to be a definite trend toward investing parts of the engineering profession with a language all their own. Should the trend continue, it is possible that an Electronics Engineer could expound for hours and a Mechanical or Electrical Engineer fail to understand him.

Language is a tool . . . a means of expression meant to be understood by everyone who speaks the language. Let's write as we speak in conversation then, and speak as though we really wanted to be understood by everybody.

Why exclude people from our conversation?



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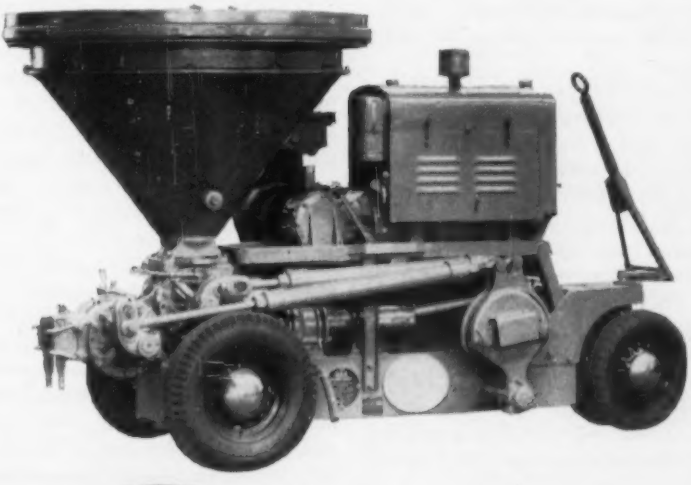

CHAIN BELT COMPANY REX CONSTRUCTION MACHINERY

Who's Who This Month

George L. Kalousek, Curtis W. Davis, Jr. and William E. Schmertz

"An Investigation of Hydrating Cements and Related Hydrous Solids by Differential Thermal Analysis," p. 693, reports experimental work at the Univ. of Toledo by the authors.

George L. Kalousek joined the research department of Owens-Illinois Glass Co. in 1944 to conduct research on insulating materials. Included in these investigations was a comprehensive study of the system lime-silica-water. He was appointed part-time lecturer in Chemistry at the Univ. of Toledo in 1945 and in this capacity began the direction of research fellowships in silicate chemistry established by the University in 1947. The present report gives results of the work of the first two recipients of the fellowships.

From 1930 to 1944 Mr. Kalousek was engaged in research on the physical and chemical properties of cements and concrete at the National Bureau of Standards. These studies have been reported in 15 publications.

Mr. Kalousek graduated from the Univ. of Idaho in 1930 with a B.S. in Ch.E. M.S. and Ph.D. degrees in Chemistry were received in 1936 and 1941, respectively, from the Univ. of Maryland.

Curtis W. Davis, Jr. received the B. Engr. degree from the Univ. of Toledo in 1947. His education was interrupted from 1943 to 1946 by service in the U. S. Navy including 2 years as an officer on an attack transport in the Central Pacific. In late 1948 he completed his studies for the M. S. degree in Chemistry at the Univ. of Toledo and then took a position in the Technical Dept. of the Libbey-Owens-Ford Glass Co.

William E. Schmertz graduated from the Univ. of Iowa with the B. S. degree in Chemistry in 1941. During the next 5 years he served in the U. S. Army, terminating his service with the rank of Major in Chemical Warfare. He completed his studies at the Univ. of Toledo for the M. S. degree in Chemistry in late 1948 and since then has been in the technical service of the Libbey-Owens-Ford Glass Co.

Frank R. Beyer

In "Stresses in Reinforced Concrete due to Volume Change," p. 713, Frank R. Beyer reports research in the Engineering Mechanics Dept., Wayne Univ., where he has been an instructor for the past two years.

Mr. Beyer received his B.S. in C.E. at Wayne Univ. in 1947 and is pursuing his graduate work there. He is a member of Sigma Xi, Society for Experimental Stress Analysis and the Engineering Society of Detroit.

Morgan B. Klock

Eastman Kodak Co. production processes sensitive to foreign materials make durable, dust-free concrete floors a necessity. From experience under these exacting conditions, Morgan B. Klock, structural engineer for the Eastman company since 1935, gives the essential requirements for good concrete floors in "Monolithic and Bonded Floor Finishes," p. 725.

Mr. Klock has been with Eastman since 1919 when he was released from the army after World War I. He has been concerned with structural design and details of industrial buildings, mostly of reinforced concrete averaging about 15,000 cu yd per year with peak production of 1000 cu yd per week. He was for a time chairman of ACI Committee 619—Wearing Surfaces for Heavy Duty Floors.

C. O. Crane

C. O. Crane describes site conditions and problems encountered in dam con-

struction in "Pumped Concrete for Enders Dam Spillway," p. 733. He has been field engineer on this Bureau of Reclamation project since 1945. From 1942-45 he was head of the Public Works Planning Section, Bureau of Yards and Docks, Navy Dept., Washington, D. C. with the rank of Lt. Commander, C.E.C.

Mr. Crane joined the U. S. Bureau of Reclamation in 1934, first as a concrete inspector, and later made temperature studies of concrete, investigated the use of vacuum processed concrete and developed an absorptive concrete form lining.

He is a 1937 graduate of Utah State College and holds M.S. and C.E. degrees from the Univ. of Colorado.

C. W. Hiner

C. W. Hiner, registered professional engineer in Nebraska, died at his home in Lincoln on April 11.

He was district engineer of the Omaha office of P.C.A. and had been with the association for 21 years. He was a member of ACI, A.S.C.E., the Omaha Athletic Club, Chamber of Commerce and the University Club of Lincoln.

Visitors at ACI headquarters

Dan H. Pletta, chairman of ACI Committee 314—Rigid Frame Bridges, who is head of the Dept. of Applied Mechanics at Virginia Polytechnic Institute made a brief call at ACI headquarters, May 20. He was accompanied by J. B. Eades and Dan Frederick also of V.P.I. All were attending the meeting in Detroit of the Society for Experimental Stress Analysis.

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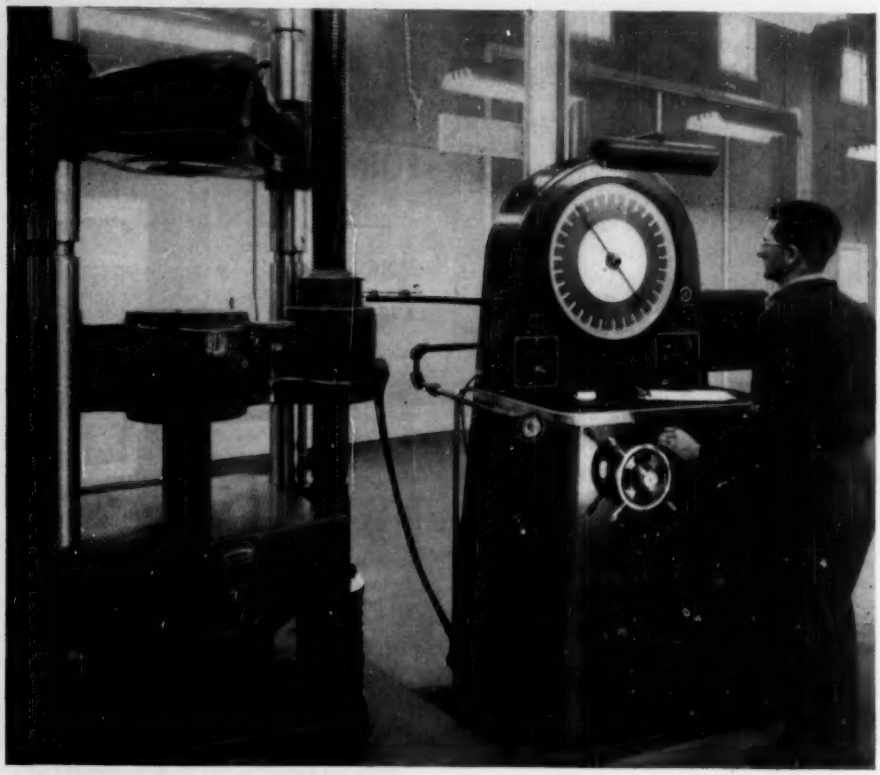

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New Members

Of the 78 applications approved for membership for the month of April there were 50 Individual, 2 Corporation, 9 Junior and 17 Students.

Top membership state for the month, providing 9 new Members, was California with New York close behind with 7 new applicants.

The membership total April 30, 1949 became 4714.

Alabama

Seale, Thomas, (Indiv.) 503 Southmont Drive, Montgomery, Ala.

California

Bryant, L. N., (Indiv.) c/o Pacific Portland Cement Co., San Juan Bautista, Calif.

Courteney, A. A. Jr., (Indiv.) Pacific Portland Cement Co., 417 Montgomery St., San Francisco 6, Calif.

Gross, Harold Dean, (St.) 1116 Carmelina Ave., Los Angeles 24, Calif.

Kuyper, William E., (Jr.) 1710 S. Sixth St., Alhambra, Calif.

Monismith, Carl L., (St.) 367 Faxon Ave., San Francisco 12, Calif.

Paff, Lloyd R., (St.) 608 Steiner St., San Francisco 17, Calif.

Peterson, E. A., (Indiv.) P. O. Box 1372, Ventura, Calif.

Rothgery, Lee J., (Indiv.) Mariposa Road, Lafayette 4, Calif.

Stickney, Charles Elinore, (Indiv.) 1725 Silver Lake Blvd., Los Angeles 16, Calif.

Colorado

Sreenivasan, A. R., (Indiv.) 1129 Pennsylvania St., Denver, Colo.

Wood, Arthur G., (Indiv.) 3158 W. 8th Ave., Denver 4, Colo.

Florida

Edelblut, Walter J. Jr., (Indiv.) P. O. Box 2054, Ft. Myers, Fla.

Illinois

Beaven, L. W., (Indiv.) 5425 Windsor Ave., Chicago 30, Ill.

Lee, Chin, (St.) Dept. of Civil Engineering, University of Illinois, Urbana, Ill.

Thompson, John P., (Indiv.) 33 W. Grand Ave., Chicago 10, Ill.

Whitehurst, Eldridge A., (Indiv.) 33 W. Grand Ave., Chicago 10, Ill.

Indiana

Longardner, Robert Loren, (St.) 611 N. 7th St., Lafayette, Ind.

Iowa

Long, Joseph D. Jr., (Indiv.) 817 Kingsley, Waterloo, Ia.

Massachusetts

Cinik, Mehmet Mesut, (St.) MIT Dormitories, Cambridge 39, Mass.

Fishman, Philip, (Indiv.) 7 Water St., Boston 9, Mass.

Michigan

Ali, Abdul Karim A., (St.) 1603 S. University, Ann Arbor, Mich.

Chao, Huang, (St.) 429 S. Division, Ann Arbor, Mich.

Cummins, Donald Kenneth, (St.) 335 Ashland Ave., Detroit 15, Mich.

Detroit Testing Laboratory, (Corp.) 554 Bagley Ave., Detroit 26, Mich.

Knoepffler, Aldo E., (St.) 130 Tyler East Quad., Ann Arbor, Mich.

Minnesota

Andersen, R. A., (Indiv.) 213 City Hall, Bldg. Inspector, Dept. of Bldgs., Minneapolis, Minn.

Missouri

Lerret, James C., (St.) 26 "T" St., Stadium Court, Columbia, Mo.

Nebraska

Hartline, Richard S., (Jr.) 1709 Jackson St., Omaha 2, Nebr.

Howell, Standley L., (Indiv.) 4939 Pine St., Omaha 6, Nebr.

Vrana, Irvin J., (Indiv.) Vrana Construction Co., 1956 S. 15th St., Omaha, Nebr.

Nevada

Cann, Elliott R., (Indiv.) P. O. Box 226, City of Reno, Reno, Nev.

Morrison, F. H., (Indiv.) Materials & Research Engineer, Dept. of Highways, Carson City, Nev.

Ready Mix Concrete Co., (Corp.) P. O. Box 1408, Reno, Nev. (John E. Yoxen)

New Jersey

Douglas, Damon G., (Indiv.) Damon G. Douglas Co., 605 Broad St., Newark 2, N. J.

Hendrickson, John G., Jr., (Indiv.) School of Engineering, Princeton University, Princeton, N. J.

New York

Colle, E. Richard, (St.) 5 Ridgewood Rd., Ithaca, N. Y.

Haddock, Gerald T., (Indiv.) 247 Park Ave., New York 17, N. Y.

Lamoreaux, Clarence, (Indiv.) 86 Long Ave., Hamburg, N. Y.

Ruth, Joseph F., (Indiv.) c/o Western Electric Co., Inc., 165 Broadway, New York 6, N. Y.

Unver, Mehmet, (Indiv.) 82-29 164th St., Jamaica 2, N. Y.

Van Name, J. E., (Indiv.) c/o Pittsburgh Testing Laboratory, 421 Canal St., New York 13, N. Y.

Zeigler, George A., (Indiv.) 7 Hanover Square, New York 5, N. Y.

North Carolina

Gordon, Olin Max, (St.) Rt. 6, Monroe, N. C.

Pennsylvania

Goldstein, Leo, (Indiv.) 6629 Wayne Ave., Philadelphia 19, Pa.

Horn, Samuel J., (Indiv.) 103 W. Broad St., Bethlehem, Pa.

Johnson, Forrest L., (Indiv.) 509 Highview Rd., Pittsburgh 16, Pa.

Meyers, Jacques Decker, (Jr.) 21 Highland Drive, Highland Village, Harrisburg, Pa.

Raj, P. V., (Indiv.) Room 601, North Office, Capitol Bldg., Harrisburg, Pa.

South Carolina

Douglas, Robert S. Jr., (St.) The Citadel, P. O. Box 1861, Charleston, S. C.

Texas

Barr, Harold A. Jr., (St.) 3833 Procter St., Port Arthur, Texas

Murlin, John A., (Indiv.) 2505 Maple Ave., Dallas, Texas

Red, David Douglass, (Indiv.) 1112 Eayle, Houston 4, Texas

Rolfe, Robert L., (Indiv.) 2204 Griffin St., Dallas 2, Texas

Sheehan, John P., (Indiv.) P. O. Box 601, El Paso, Texas

Washington

Reilly, James M., (Indiv.) 426 N. Meridian, Puyallup, Wash.

Wisconsin

Holway, Alvah S., (Indiv.) 5410 W. Louise Pl., Milwaukee 10, Wis.

Wyoming

Hulme, Robert E., (Jr.) 709 Grand Ave., Laramie, Wyo.

John, Harry M., (St.) 418½ S. Sixth, Laramie, Wyo.

Lang, Paul Arthur, (Jr.) 215 W. 9th St., Cheyenne, Wyo.

Peterson, Frederick A., (Jr.) 1207 Cosgriff Court, Cheyenne, Wyo.

Hawaii

Loo, B. C., (Indiv.) P. O. Box 3619, Honolulu, T. H.

Puerto Rico

Leyenberger, Lawrence Alden, (Indiv.) Cervantes 10, Condado, Santurce, Puerto Rico

Ramirez, David S. Castro, (Jr.) Calle Bouret No. 505-1B, Parada 26, Santurce, Puerto Rico

Rodriguez, Rafael, (Jr.) 1365 Luchetti St., Santurce, Puerto Rico

Canada

Majoros, Steve, (Indiv.) 1629 St. Elizabeth St., Montreal 18, P. Q., Canada

Costa Rica

Hernandez R., Juan Ramon, (St.) Apartado 1404, San Jose, Costa Rica

England

Kukielski, K., (Indiv.) 40, Hereford Rd., London W. 2, England

Szczerbaniewicz, S. J. M., (Indiv.) 33, Lexham Gardens, London W. 8, England

India

Gopal Midha, Chandhri Nand, (Indiv.) Retds. S. Ein. I. Br. D. E. n E. Br P. 152, Benmore No. 2, Simla East, India

Ireland

Hughes, Andrew J., (Indiv.) 7 Camden Bldg's. off Camden St., Dublin, Ireland

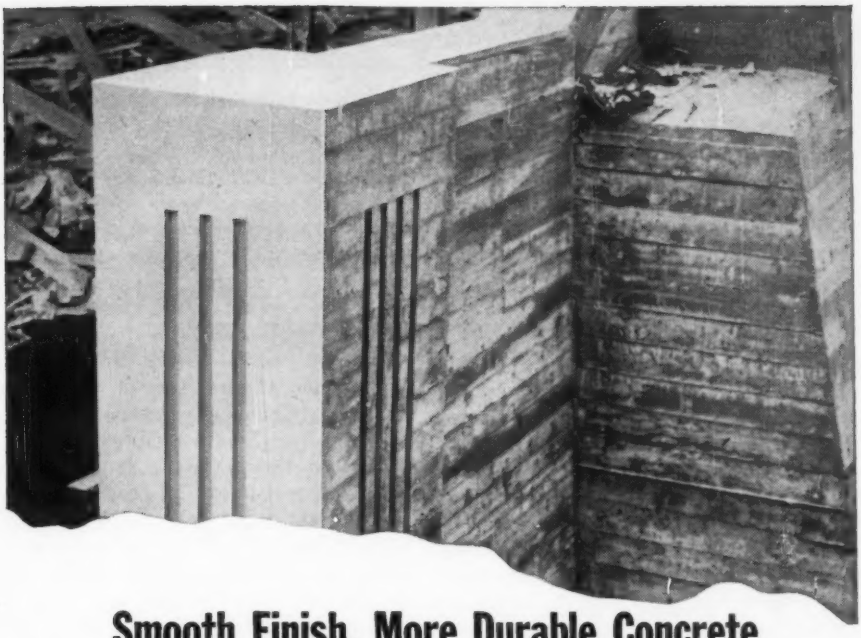
Israel

Adler, Jacob, (Indiv.) c/o Horman & Einziger, 5 P. Smolenskin St., Tel Aviv, Israel

Malaya

Cumming, J., (Indiv.) Executive Engineer Marine, Public Works Dept., Singapore, Malaya

Continued on p. 21



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Positions and Projects — ACI Members

Iowa Engineering Society meets ACI Secretary

Harvey Whipple, ACI Secretary-Treasurer, was the guest speaker at an informal meeting of the Iowa Engineering Society at the Memorial Union, Iowa State College, May 2. Introduced by ACI President Gilkey, Mr. Whipple told something of the activities of the Institute and pointed out the remarkable diversity of membership, consisting as it does of cement chemists and manufacturers; structural engineers and designers, architects and builders, aggregate producers and contractors; inspection and testing engineers and members of the technical staffs of various divisions of government with a liberal sprinkling of engineering educators and college research personnel. He emphasized that ACI is strictly a professional rather than a trade organization, being what might be termed a "community-of-interest" group with members throughout the world, bound together by a common interest in concrete. He mentioned some ways in which ACI's work might be broadened to give more nearly adequate coverage of the many diverse areas of interest.

Other speakers discussed cement supply and problems related to shortages, the Iowa Highway Commission construction program, status of building plans and the resumption of concrete short courses interrupted by the war. Dean Frank Kerekes was one of the contributors to the discussion.

ACI Committee 505 reactivated—members appointed

ACI Committee 505—Reinforced Concrete Chimneys, has been reactivated with E. A. Dockstader of Stone & Webster Engineering Corp., Boston, as chairman. At Mr. Dockstader's suggestion the following men have been appointed as a nucleus of the new committee: Howard M. Estes, also of Stone & Webster; O. G. Julian, Jackson and Moreland, Engineers, Boston; Miles N. Clair, Thompson & Lichtner Co., Inc., Brookline, Mass.; K. B. Foster, Rust Engineering Company, Pittsburgh; and George Shervington, Consolidated Chimney Co., New York. The latter three accepted appointment too late to have their names included in the 1949 Directory.

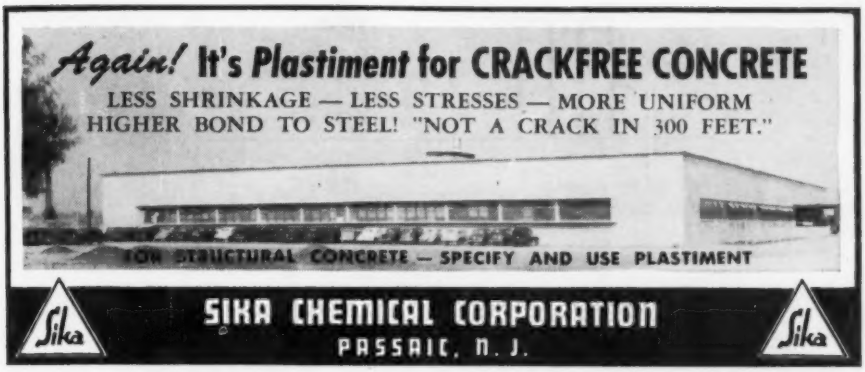
The former committee prepared a "Proposed Standard Specification for the Design and Construction of Reinforced Concrete Chimneys" which was announced in the March-April 1934 JOURNAL and later made available in 1936 in a lithoprint edition at 75¢ per copy. The report has never been adopted as a standard but has been in continuous moderate demand requiring several reprintings. Interest in late years indicates that the need for this standard is still great and therefore the committee has been reactivated. Mr. Dockstader also served as author-chairman of the first committee report.

Visual education committee named

ACI President Gilkey has named Harmer E. Davis, chairman, and Harry C. Delzell, Robert F. Blanks and Harrison F. Gonnerman members of a committee of the Board of Direction to study and report whether or not and how ACI might sponsor motión picture films as an instrument of visual education on good job practice in making concrete. The idea was suggested to ACI by past president Raymond E. Davis at a small group meeting in St. Louis last fall and later to the Board of Direction at its meeting last February. The committee also is to study possibilities for financing, production and distribution.

Savage receives Washington award

John L. Savage, internationally recognized hydraulic engineer, received the Washington Award for "unselfish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public service devoted to the creation of monumental hydraulic structures utilizing natural resources." The award was given April 20 by the Washington Award Commission representing engineering societies with a total membership over 50,000.

Mr. Savage was honored for responsibility for more than 300 dams and reclamation and power plant projects as a consulting engineer and as chief designing engineer for the U. S. Bureau of Reclamation. Best known projects are Hoover, Norris (TVA), Wheeler (TVA) and Grand Coulee Dams.

Since 1945, Mr. Savage has spent most of his time as consultant on hydraulic projects for foreign governments, notably India and China.

An ACI member since 1928, Mr. Savage was awarded the Henry C. Turner Medal in 1946 "in recognition of long and distinguished service in the design of hydraulic structures including some of the world's most notable dams."

National Research Board award to Scholer

Prof. C. H. Scholer, head of the applied mechanics department, Kansas State College, Manhattan, Kan., has received the distinguished service award from the highway research division of the National Research Board, Washington, D. C. This is only the second such award made by the Board. Professor Scholer pioneered studies on concrete durability. As early as 1926 A.S.T.M. published his study of accelerated freezing and thawing tests on concrete; later reports have been published by ACI and the Highway Research Board.

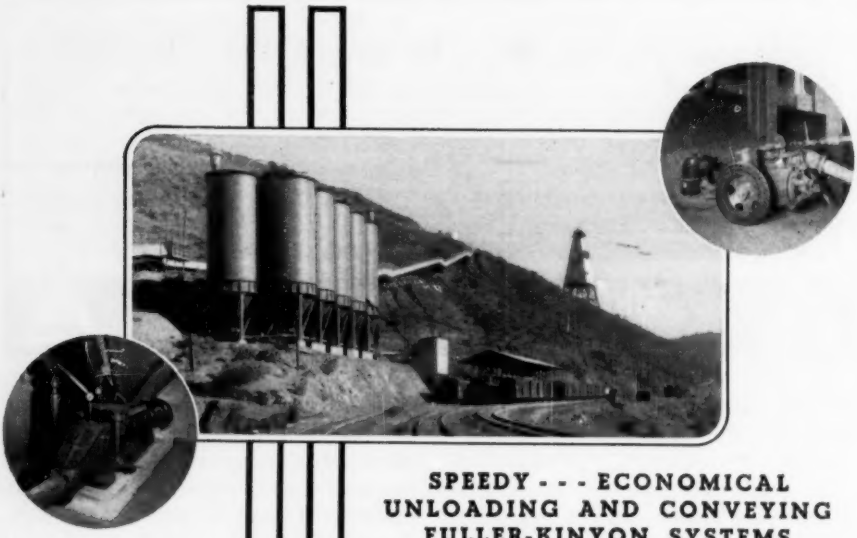
Design method presented at A.S.C.E. meeting

A method for calculating varying end restraints of compression members in rigid frames was presented by Harold E. Wessman, Dean of Engineering, Univ. of Washington, and Thomas C. Kavanagh, Dept. of Civil Engineering, Pennsylvania State College, at the spring meeting of A.S.C.E. Structural Division at Oklahoma City, April 20-22.

The procedure developed by the authors, both members of ACI, is an extension of the Lundquist moment-distribution method for analyzing the stability of a structural frame.

Rothgery heads training program

Lee J. Rothgery recently became associate engineer with the Institute of Transportation and Traffic Engineering, Univ. of California, in charge of the organization of, and instruction in, the in-service training program. He spent two years with the Michigan State Highway Dept., and 20 years teaching at Michigan State College. He has engaged in private practice and contracting and is a registered civil engineer in California.



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Coff addresses joint meeting of engineers

L. Coff, consulting engineer, spoke on "Prestressed Concrete in the United States and Europe" before a May 25 joint meeting of the Structural Engineers Society of New York and the New York County chapter of the New York State Society of Professional Engineers.

Mr. Coff has been connected with work in prestressed concrete both in Europe and the United States and recently visited major prestressed concrete construction jobs in Europe to make cost comparisons with similar work in this country.

Monarch rehabilitation program

Monarch Cement Co., has announced that the first phase of its extensive construction program underway at its Humboldt, Kan. plant is scheduled for completion during 1949. The rehabilitation program, according to Walter H. Wulf, president, consists of replacing present equipment with new and more modern equipment. The first phase was started in 1946 with installations being made without interrupting production.

Improvements made include installation of a 230-ft Allis-Chalmers kiln which will eventually replace three smaller ones now in operation, a 70-ft Allis-Chalmers air-quenching clinker cooler, a new 300-ft concrete stack, two new boilers and a coal mill and complete new coal-handling equipment and accessories.

Second phase of the contemplated over-all program calls for another large kiln, an air-quenching clinker cooler and a third boiler, which, when installed, will completely replace the present kiln, cooler and boiler setup.

P.C.A. centralizes Texas activities

Portland Cement Assn. district offices in Dallas and Austin, Texas, have been consolidated into a single district office in Austin. James D. Piper, formerly P.C.A's district highway engineer in Dallas, has been appointed district engineer in charge of all P.C.A. work in Texas. He joined the association in 1937 as a structural engineer in Oklahoma City. In 1939 he was transferred to the Austin district as a structural engineer for the northern half of Texas.

In 1942 he accepted a commission in the U.S. Navy Civil Engineer Corps and served in the South Pacific with the 61st U.S. Naval Construction Battalion, which handled airfield, road, wharf and general building construction. Separated from naval service as a Lt. Commander in 1945, he returned to the association as a structural field engineer with headquarters in Houston.

Prior to joining P.C.A., Mr. Piper spent two years with the state of Oklahoma as an architectural engineer. He was graduated from Oklahoma A. & M. College in 1934 with a B.S. degree in architectural engineering, and did graduate work in structural engineering at the Univ. of Illinois in 1935. He is a licensed architect and a registered professional structural engineer in Texas. He is also an associate member of A.S.C.E.

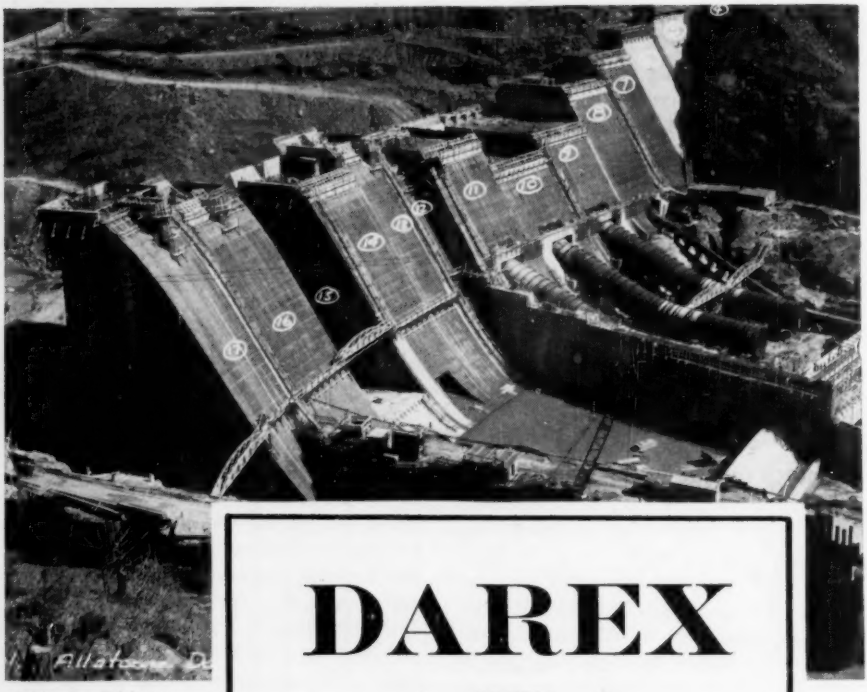
Charles A. Clark, who has been in charge of P.C.A. activities in other than the highway paving and bridge fields, will continue as office engineer in the newly consolidated Austin office. Mr. Clark, a civil engineering graduate of the Univ. of Missouri in 1912, has been with the association for more than 27 years.

Friel named to Drexel alumni post

Francis S. Friel, president and treasurer of the Philadelphia firm of Albright & Friel, Inc., has been named to the chair of the Alumni Section of the Drexel Institute of Technology expansion program. Mr. Friel recently served as president of the Philadelphia section.

Cates joins Navy C.E.C.

Vincent K. Cates, formerly manager of the Logan International Airport at East Boston, Mass., will join the Navy Civil Engineer Corps in a civilian status.



Allatoona Dam. Cartersville, Georgia. Courtesy U S Corps of Engineers.

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Admiral Cotter named president of Merritt-Chapman & Scott Corp.

Carl Henry Cotter, Rear Admiral (CEC) U. S. Navy (Retired) was named president of Merritt-Chapman & Scott Corp. by its board of directors, effective May 9, 1949. Admiral Cotter's naval service dated from 1918 when he entered as an assistant civil engineer. He was director of the Pacific Div., Bureau of Yards and Docks from 1943-45. He was awarded the Distinguished Service Medal, the Legion of Merit and numerous other decorations.

He is a member of ACI, A.S.C.E. and the Society of American Military Engineers.

Strudy new Dravo director

Howard H. Strudy, general manager of the Contracting Division of the Dravo Corp., Pittsburgh, Pa., has been named one of the new Dravo directors. Starting as a field engineer with the organization in 1936, Mr. Strudy was appointed to his present position in 1947.

Frech retires

Harry E. Frech, district engineer in the St. Louis office of the Portland Cement Assn., has retired after 30 years with the association, which he joined in 1919 as a field engineer in Missouri. He became district engineer in 1920 and served continuously in that capacity until his retirement except for an eight months' leave of absence in 1941 to serve with the U. S. Army as chief zone engineer for the 7th Construction Zone of the Quartermaster Corps at Omaha.

Renamed to registration board

Asa G. Proctor, of Woodland, Calif., has been reappointed to a four-year term on the State Board of Registration for Civil and Professional Engineers.

Palmer joins U.S.B.R.

Fred I. Palmer has joined the staff of the U. S. Bureau of Reclamation and is working on highway relocation at the Colorado Big Thompson project.

Progress Report of ACI Committee 315— Detailing Reinforced Concrete Structures

[With the resignation of the late A. J. Boase as chairman, Committee 315 was reorganized with membership shown below. Chairman Reese reports the following recommendations of a majority of his committee as errata—a considerable number of proposed revisions of the ACI Detailing Manual first as published in 1946 and reissued in Jan. 1949 in a third printing. The second printing in April 1947 was identical with the first printing except for appropriate changes on the title page. The third printing, 1949, is likewise identical with both previous printings, except:

1) the "proposed" Manual is now a full Standard (ACI 315-48) following action at the February 1948 convention and letter ballot approval June 29, 1948.

2) appropriate changes on title page and cover.

3) a progress report of reorganized Committee 315, under the chairmanship of Mr. Reese, was added in the front of the book indicating recommendations for changes for purposes of clarification and correction of editorial and printing errors. The progress report is published here

for the information of members. The Standard was adopted without change in all pages following the title page. This **progress report** carries only the authority of Committee 315, in pointing out future changes which it deems desirable; it has no authority as a Standard. —Editor's note].

ERRATA—Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI Standard 315-48)—Recommended by Committee 315,

RAYMOND C. REESE, Chairman

EARL P. ALLABACH
FRANK H. BEINHAUER
RENE L. BERTIN
EVERETT E. EBLING
FRANK KEREKES
T. D. MYLREA
FRED L. PLUMMER
E. E. RIPPSTEIN
JOHN F. SEIFRIED

p. 7, par. (d), line 1—Interchange "show" and "clearly".

p. 7, par. (d), line 3—After "$\frac{1}{8}$" = 1'0"", add asterisk with footnote: "In the case of large structures, it is recommended that they be divided into separate areas of such size as can be shown to $\frac{1}{8}$" scale on a single sheet, and add a small-scale key or location plan just above title block, with the particular area shown on the sheet shaded or cross-hatched."

p. 7, par. (d)—Under heading "Sections", add "$\frac{1}{4}$" = 1'0"".

p. 10, par. (e), par. 2, line 1—After "Table 9", insert "Page 30".

p. 10, par. (f), line 14—After "Table 8", insert "Page 29".

p. 12, par. (j), par. 1—At the end of the first paragraph, after "Drawing 2", add: "or avoid splicing by using straight top and bottom bars."

p. 12, par. (j), par. 2, line 6—Delete second "t" in "conditions".

p. 12, par. (l)—Immediately above Article 108, after "(3)", add: ", or with slab soffit flush with underside of top flange of beam to simplify formwork."

p. 15, dwg. 4, col. 6—After heading "Type", add asterisk with footnote: "See Drawing 12 for types of bends."

p. 16, par. (a), line 2—Change "a builder's" to "the builder's".

p. 16, item 14, line 3—Change "inch" to "inches".

p. 18, art. 204, par. 2, line 10—Delete "only about half" and substitute "considerably less than".

p. 18, art. 204, par. 2, line 11—Change "many" to "may".

p. 18, art. 204—Add a note, "The method of pricing extras has been revised and details should be obtained from local fabricators."

p. 19, art. 303, par. 1, line 5—Change "d" to "g" in "longitudinal".

p. 20, art. 307, par. 1, line 3—Delete the second "e" in "transverse".

p. 21, table 2—After last two items, add asterisks with a footnote: "Usually furnished in galvanized wire."

p. 25, table 7—Immediately under the heading "Number of Bar Diameters . . . ", add: "Increase lap 25 percent if plain round bars are used."

p. 33, dwg. 15—The following changes have been agreed upon (there will be others): Move north arrow from lower righthand corner of framing plan to upper lefthand corner of sheet. (North arrow should be handled similarly on all appropriate drawings.) On "Typical Floor Construction", show temperature reinforcement with a full line instead of dashed.

In "Notes", in Note 3, line 1, change "12" to "8"; also in line 2, after "reinforcement", add "with spacer bars of $\frac{1}{4}$ ϕ @ 4'2" c/c"; and on the same line, change "9/12" to "8/12".

p. 34, dwg. 16—(See note re Drawing 15) Change number and spacing of temperature steel in each panel of the floor framing to agree with 8″ spacings instead of 12″.
Add detail of spacer rods "1/4 ϕ @ 4′2″ c/c".
On "Typical Beam" detail, at top, change "1/4 ϕ @ 12" to "1/4 ϕ @ 8".
On "Section Y-Y", change "1/4 ϕ @ 12" to "1/4 ϕ @ 8".
On "Section Y-Y", show temperature steel with full lines.
p. 34, dwg. 16—"(The designation of temperature reinforcement on the framing plan will eventually be changed to agree with the corrections)".
p. 34, par. (b), lines 3 & 4—Interchange "definitely" and "to".
p. 38, dwg. 18, par. (b)—At end of par. (b), add new paragraph: "If bands are marked with even numbers one way and odd the other or with the first letters of the alphabet one way and the last letters the other way, there is some convenience in distinguishing which ways the bands run."
p. 42, dwg. 22—In "Beam Schedule", under heading "Stirrups", in Column 3, change "5′9″" to "5′5″" in five places; namely, beams 2B1, 2B2, 2B6, 2B23, and 2B24.
Change "4′7″" to "4′3″" in eight places; namely, beams 2B3, 2B4, 2B5, 2B7, 2B8, 2B9, 2B10, and 2B11.
Same place, in fourth column, change "27" to "24" for beams 2B1 and 2B2.
p. 43—Add paragraph (d): "(d) Special Steel. A note is included that the threaded dowels, bar couplings, and bolts are to be furnished with the column reinforcement. Unless this is done, the fabricator will not supply them, since they are not standard reinforcement. The contractor is apt to overlook ordering them until they are needed on the job."
p. 43, dwg. 23—(See notations under Drawing 15)
Just to the right of Footing 3-3A on the "Foundation Plan", add Section Marks "D-D".
Below and to the right of the "Foundation Plan", add Section D-D.
At top of sheet, "Detail X", after the notation "1″ plain round x 2-6 threaded dowel", add "furnished with column reinforcement".
On "Foundation Plan", between Columns 25 and 26, draw a line across the ramp and mark "expansion joint in walls and slab".
p. 44, dwg. 24—On "Foundation Plan", between Columns 25 and 26, draw a line and add "expansion joint in walls and slab".
On "Section B-B", between building wall and ramp, draw a line and add "expansion joint in walls and slab".
In "Corner Detail" at lower lefthand side of sheet, in note, third line, change "72" to "79".
On "Foundation Plan", to the right of Columns 3 and 3A, add Section Marks "D-D".
p. 44—At end of explanation, add a paragraph: "If Column Schedule is not available to bar setter for placing column dowels, add a line in the schedule reading 'set inside of . . . ' and give the size of rectangle that will put the rods inside of the column ties above, or else add a line headed 'Pier Size' and let the bar setter come well inside of this."
p. 45, dwg. 25—In "Column Schedule", last two columns, Piers 21 to 26, inclusive, add descriptions of threaded dowels as mentioned before on Drawing 23, p. 43.
At note under the "Column Schedule", at the end of Line 1, add: ", except at bends in column verticals, where they shall be 3″ c/c."
p. 46, dwg. 26—Change designation of reinforcing steel from dashed lines to full lines on all views.
p. 52, dwg. 32—In the title block, change "Digester" to "Clarifier".
p. 53, dwg. 33—In the title block, change "Digester" to "Clarifier". Further down in the title block, change "Archt's" to "Engr's".
p. 55, dwg. 35—In title block, change "Archt's" to "Engr's".

Honor Roll

February 1, 1949 to April 30, 1949

There is little change in the standing of our top "Member Getters" for the period ending April 30, 1949.

Prof. Newlin D. Morgan, Univ. of Illinois, remains in the number one position with 42½ credits. Prof. Newlin D. Morgan, Jr., Univ. of Wyoming, moved up to tie with Prof. F. N. Menefee, Univ. of Michigan, for the number two spot. Each has ten credits.

Newlin D. Morgan	**42½**
F. N. Menefee	**10**
Newlin D. Morgan, Jr.	**10**
Jaime Alberto Mitrani	**6**
J. Vicente Orozco	**4**
Jose Antonio Vila	**4**
R. H. Sherlock	**3½**
Raymond E. Davis	**3**
John J. Hogan	**3**
H. D. Jolley	**3**
J. W. Kelly	**3**
Leon Venegas	**3**
Stanton Walker	**3**
Roger H. Corbetta	**2½**
Thomas E. Stanton	**2½**
Paul W. Abeles	**2**
H. J. Gilkey	**2**
Henry E. Griset	**2**
Emory E. Johnson	**2**
R. R. Kaufman	**2**
Karl W. Lemcke	**2**
A. J. McElrath	**2**
William T. Neelands	**2**
William D. Nowlin	**2**
Roberto Barrilas F.	**1½**
R. A. Caughey	**1½**
S. J. Chamberlin	**1½**
H. F. Gonnerman	**1½**
Elmo C. Higginson	**1½**
Barton Jones	**1½**
D. H. Pletta	**1½**
Edward W. Scripture, Jr.	**1½**
Thomas C. Shedd	**1½**
Jerome O. Ackerman	1
William F. Anderson	1
Samuel Baker	1
J. A. Bakker	1
Charles F. Ball	1
John H. Bateman	1
J. F. Baxter	1
Oscar Benedetti	1
Robert F. Blanks	1
Hans H. Bleich	1
Carlos Mora Bravo	1
Milton Brumer	1
R. E. Copeland	1
Alirio Cornejo	1
W. S. Cottingham	1
G. H. C. Crampton	1
Ray A. Crysler	1
Harmer E. Davis	1
Peter J. Doanides	1
A. T. Docal	1
W. H. Droege	1
Belmon U. Duvall	1
Harlan H. Edwards	1
J. M. Faircloth	1
F. L. Fitzpatrick	1
Ambrosio R. Flores	1
Robert W. Freeman	1
Carmelo Galindez	1
Gerald K. Gillan	1
Ali N. Gokyigit	1
Armour T. Granger	1
K. Hajnal-Konyi	1
Robert J. Hansen	1
Percy Howat	1
J. W. Hubler	1
Alberto Dovali Jaime	1
J. P. Jain	1
V. P. Jensen	1
Edgar R. Kendall	1
K. V. Kini	1
Ralph W. Kluge	1
George E. Large	1
Leslie D. Long	1
Glenway Maxon	1
James A. McCarthy	1
F. R. McMillan	1
M. J. McMillan	1
Carl A. Menzel	1
Jesus Mesa	1
Robert B. B. Moorman	1
Robert Morris	1
George G. Nichols	1
LeRoy T. Oehler	1
Jose F. Perez	1
Charles Peterson	1
W. S. Powell	1
Emil Praeger	1
Raul Miguel Rivero	1
Roy T. Sessums	1
Saul Shaw	1
Ranbir Singh Shrikant	1
Richard Skalak	1
Luis G. Solano A.	1
Frederick E. Springate	1
Alois Starkmann	1
W. B. Summer	1

Manabu Tanaka 1
Maurice Taylor 1
T. W. Thomas 1
G. E. Troxell 1
I. L. Tyler 1
Oscar J. Vago 1
Joseph J. Waddell 1
Bertrand H. Wait 1
Stanton Walker 1
John M. Wells 1
Frank W. Wheeler 1
George Winter 1
Donald K. Woodin 1

The following credits are, in each instance "50-50" with another member.

C. Harold Barcus
Charles H. Barker
Frank Baron
J. F. Barton
J. E. Bates
Aaron H. Bauman
E. W. Bauman
R. W. Brandley
A. H. Brewer
Harold W. Brewer
A. R. Brickler
Robert G. Burnett
R. Y. Bush
J. A. Carrick
Napoleon V. Campomanes
Harold S. Carter
Arthur P. Clark
R. D. Conrow
Leo H. Corning
Lauro M. Cruz
W. A. Davis
Harry Delzell
A. H. Douglas
W. J. Emmons
J. L. Faisant
Phil M. Ferguson
Benjamin Forsyth
Adly W. Gindy
A. W. Haddow
J. R. G. Hanlon
Adli Milik Hanna
M. J. Hawkins
Albert C. Hooke
Bruce Jameyson
Frank Kerekes
Max Krueger
Henri Marcus
Hudson Matlock
Robert L. Mauchel
J. W. McBurney
T. R. McCullough
Jack Robert Miller
William S. Moore
Wm. Morris
Philip M. Noble
George L. Otterson
Chester B. Palmer
Douglas E. Parsons
Robert J. Perrich
Alan H. Pilling
Warren Raeder
Irvin S. Rasmusson
Frank E. Richart
John A. Ruhling
Fred H. Ryan
Jack S. Short
Ruben Ruiz Silva
Charles A. Snyder
H. F. Sommerscheild
G. W. Stokes
E. O. Sweetser
Anton Tedesko
William H. Thoman
J. Trueman Thompson
E. W. Thorson
LeRoy A. Thorssen
Homer Thrall
Fehiman Tokluoglu
Gregory Tschebatarioff
John Tucker, Jr.
F. E. Votaw
W. Wayne Wallace
M. H. Walters
David Watstein
J. C. Watt
W. R. Waugh
Carl Weber
Charles S. Whitney
H. F. Williams
Paul C. Winters
Thomas Worcester
K. W. Wright
Alexander H. Yeates
Robert Zaborowski

New Members

Continued from p. 10

Mexico

Saldivar, Jose, (Jr.) Rio Amur No. 13, Col. Cuauhtemoc, Mexico D. F., Mexico

Netherlands

Haas, A. M., (Indiv.) c/o Nedam, 135 Binckhorstlaan, The Hague, Netherlands

Philippines

Bautista, Mariano B., (Indiv.) Barasoain, Malolos, Bulocan, Philippines

Southern Rhodesia

Haviland, John Haviland, (Indiv.) P. O. Box 1874, Bulawayo, Southern Rhodesia

Yugoslavia

Markovich, Miroslav, (Indiv.) Ul, Bircaninova 28/II, Belgade, Yugoslavia

Tools, Materials, Services

Under this heading note will be made from time to time of producer literature of presumed technical interest (and available from its source for the asking) to ACI users of tools, equipment, materials, accessories and special services.

Portland cement specifications

Cembureau, The Cement Statistical and Technical Bureau, Malmö, Sweden, has recently published "Tabular Review of the Portland Cement Standards of the World." It presents in an easily comprehensible arrangement the main features of standard specifications for portland cement, both ordinary, rapid hardening and other types, as ruling at the end of 1948 in 34 countries. Reference is also made to 17 countries having no standard of their own, but employing or accepting one or more of internationally recognized specifications. Requirements for chemical composition, fineness, setting time and soundness are stated in reasonable detail, and also strength requirements together with methods of preparing test specimens and specification of sand required for strength tests. Strength figures are stated both in metric and British units.

The booklet, prepared in English by E. A. Tolfsby may be obtained from Cembureau, The Cement Statistical and Technical Bureau, Malmö, Sweden. Price $2.50.

Rock cork insulation

Rock cork is a mineral insulation said to be especially valuable for refrigeration service in the food and beverage industries. A four-page folder provides essential data on size, temperature limits, conductivity, moisture absorption and similar information. Also included are suggested uses for this material in sheets, lagging and pipe insulation.

Copies of this brochure may be obtained from Johns-Manville, 22 East 40 Street, New York 16, N. Y.

DON'T FORGET—
there will be no Journal issues in July and August

SYNOPSES of recent ACI Papers and Reports

Institute papers of this JOURNAL Volume 20 are currently available at prices indicated. Please order by title and title number.

RECOMMENDED PRACTICE FOR WINTER CONCRETING METHODS (ACI 604-48).....45-1

Price 50 cents.

REPORT of COMMITTEE 604—Sept. 1948, pp. 1-20, (V. 45) in special covers

Supersedes 44-13.

This ACI standard establishes methods of cold-weather concreting for thin sections and mass concrete. Heating of materials, accelerators and anti-freezes, curing and temperature records during curing, subgrade (or base) preparation, protective coverings during curing, and form removal are discussed for both types of job, and preferred methods are indicated. An appendix entry outlines objectives of the special winter methods with background material which indicates the "why" of some of the recommended practices. Charts in the appendix indicate effect of curing temperature on concrete strength, and a list of 135 selected references to periodical literature on winter concreting methods is included.

RELATIVE STRENGTHS OF PORTLAND CEMENT MORTAR IN BENDING UNDER VARIOUS LOADING CONDITIONS.....45-2

Price 35 cents.

JACOB PORTER FRANKEL—Sept. 1948, pp. 21-32 (V. 45)

Assuming the behavior of standard mortar to be similar to that of plain concrete, tests were performed on 99 small mortar beams under sixth-, third- and center-point loading to verify the applicability of the statistical theory of the strength of brittle materials to concrete structures. The theory, as first developed by Weibull, is briefly analyzed, compared to the methods used by Tucker, and finally applied to the tests at hand. Agreement between the theoretical and experimental findings is so close as to warrant future testing on plain concrete specimens.

TERRAZZO AS AFFECTED BY CLEANING MATERIALS.....45-3

Price 35 cents.

D. W. KESSLER—Sept. 1948, pp. 33-40 (V. 45)

Effect of cleaning materials on terrazzo was studied using solutions of the following detergents: soda ash, trisodium phosphate and synthetic sulfonate. Seventy-six different aggregates were used in preparing terrazzo disks which were moist cured 3 months and then surface ground to form a shallow dish. Detergent was allowed to stand in dish 30 minutes before rinsing and drying (at 105 C). This testing cycle was repeated until deterioration tendencies were established. Soda ash solution proved much more destructive than the trisodium phosphate; however, up to the time of writing (after 850 cycles) the synthetic sulfonate had caused no failure.

TRANSPORTING READY-MIXED CONCRETE IN OPEN DUMP TRUCKS.....45-4

Price 35 cents.

R. A. BURMEISTER—Sept. 1948, pp. 41-56 (V. 45).

Milwaukee experience in hauling air-entraining concrete in open dump truck bodies is recorded. Slump, air content, specific weight, compressive strength and workability tests were made on concrete before and after a trial run over bumpy pavement. Some similar tests were made on concrete being placed in a Milwaukee street after open truck transportation. The author concludes that open trucks for hauling ready-mixed air-entraining concrete are satisfactory when air content is between 3 and 6 percent (5 to 6 sacks cement per cu yd) and hauling time is 45 minutes or less, provided that aggregate moisture, grading and slump are carefully controlled.

A STUDY OF ALKALI-AGGREGATE REACTIVITY BY MEANS OF MORTAR BAR EXPANSIONS.....45-5

Price 35 cents.

T. M. KELLY, L. SCHUMAN and F. B. HORNIBROOK—Sept. 1948, pp. 57—80 (V. 45)

Many types of aggregate were combined in varying amounts and sizes with high- and low-alkali cements and formed into 1 x 1 x 10-in. mortar bars. The bars were stored either at 70 F or at 100 F and their expansions measured at ages ranging from 1 month to 4 years.

In combination with high-alkali cements, opal, opaline chert and a siliceous dolomitic limestone were found to cause greatest expansion. Certain aggregates containing volcanic glasses and some natural sands and gravel also caused excessive expansion; with one exception, these sands contained small amounts of opal.

Greatly delayed expansion resulted with the very fine sizes of opal, particularly in combination with high-soda cement. Similar behavior resulted with minus No. 81 size opal and low-alkali cement with either Na_2SO_4 or K_2SO_4 additions.

Materials such as dehydrated kaolin, soda feldspar, magnesium fluosilicate, acetic acid and calcium hydroxide added in small amounts as correctives were ineffective. However, diatomaceous earth in sufficient quantity as a cement replacement eliminated expansion.

REINFORCED CONCRETE WALL AND COLUMN FOOTINGS PART I.....45-6a

Price 60 cents.

FRANK E. RICHART—Oct. 1948, pp. 97-128 (V. 45)

PART II.....45-6b

Price 35 cents.

FRANK E. RICHART—Nov. 1948, pp. 237-260 (V. 45)

This paper presents, in two parts, a report on an experimental investigation of reinforced concrete footings. It represents the first extensive study of the subject since the well-known work of Talbot in 1913. In the meantime there have been many developments in materials, as well as in design methods.

Important developments from these tests include the following: (a) Definite proof that the tensile and bond resistance depend upon the bending moment and shear found by statics by consideration of the full applied load, and not 85 percent thereof, as assumed in current building codes; (b) Hooked ends of bars showed no particular advantage in bond resistance over straight bars, particularly when deformed bars of new improved types were used; (c) Hooked bars produced very little effect on resistance to diagonal tension, much less than is commonly assumed; (d) Welded mats proved particularly effective in resisting end slip of bars; (e) Footing caps or piers are effective in reducing the amount of reinforcement and in increasing the load capacity of footings; and (f) Diagonal tension seems to be the point of weakness in current design practice. The factor of safety of thin footings in this respect appears greater than in thick footings, and is generally greater in rectangular than in square footings when the conventional methods of computation are used.

In these tests, major emphasis has been placed on the isolated column footing. Principal attention has been

given to the resistance of footings to failure by bond, diagonal tension and tension in the steel. Test footings were designed to produce these various types of failure. In addition, studies were made of the behavior of rectangular footings and footings having intermediate capitals or piers.
The tests featured measurements of tensile and compressive strains, deflections and slippage of bars. In some of the tests three grades of concrete were used, in others, five types of reinforcing bar were employed. In most of the tests, the footings were 7 ft square. They were supported on a bed of steel springs and tested in a large testing machine.
This paper merely attempts to present and interpret the test results, and does not offer recommendations as to possible changes in design procedures at this time.

LATTICE ANALOGY IN CONCRETE DESIGN.....45-7

Price 35 cents.

DOUGLAS McHENRY—Oct. 1948, pp. 129-140 (V. 45)

The lattice analogy is a scheme for solving two-dimensional stress problems in which the involved mathematical methods of the theory of elasticity are replaced by simple computations. The solid section is replaced by an equivalent lattice or framework which may be solved by methods applicable to indeterminate structures. Stress distribution in sections of complex shape and with complex loading may be determined by successive approximations which involve only substitution in simple equations. The method is illustrated by application to the problem of stresses in a deep beam with off-center loading.

STRENGTH OF PRECAST CONCRETE FLOOR JOISTS.....45-8

Price 35 cents.

JACOB FELD—Oct. 1948, pp. 141-148 (V. 45)

The strength and usability of precast concrete floor joists left in the open without any protection for a year, when a housing project was abandoned, were proved by load test on a slab section incorporating the poorest joists. This paper describes the condition of the joists, the load test results and indicates that a greater tolerance can be safely permitted in the visible defects of precast concrete joists.

EFFECT OF ENTRAINED AIR ON CONCRETES MADE WITH SO-CALLED "SAND-GRAVEL" AGGREGATES.....45-9

Price 35 cents.

PAUL KLIEGER—Oct. 1948, pp. 149-164 (V. 45)

Sand-gravel aggregates (maximum particle size ¾ in.) used with non-air-entraining cement produce concretes containing from 3 to 4 percent air. The same mixes made with air-entraining cement and the sand-gravel aggregates have air contents from 8 to 13 percent. Effect of this larger amount of entrained air is given in terms of tests of flexural and compressive strength, freezing and thawing resistance, and length changes in varying storage conditions. Effect of "sweetening" the sand-gravel by addition of 30 percent of coarser aggregate is noted.

BURNED SHALE AND EXPANDED SLAG CONCRETES WITH AND WITHOUT AIR-ENTRAINING ADMIXTURE.....45-10

Price 35 cents.

P. H. PETERSEN—Oct. 1948, pp. 165-176 (V. 45)

The physical properties of several lightweight aggregate portland cement concretes made with burned shale or expanded slag were investigated at the National Bureau of Standards. Three grades of concrete were made with each aggregate. Air-entrainment greater than 20 percent is reported for the mixtures leanest in cement, an air-entraining admixture being used to increase the workability of all but the richest concretes. Compressive, flexural and bond strength data are given as well as resistance to heat transfer, rain penetration and water penetration by capillarity. Also included are the coefficients of thermal expansion, shrinkage, and values for change in length due to wetting and drying.

CEMENSTONE PRECAST CONSTRUCTION.....45-11

Price 35 cents.

A. C. GRAFFLIN—Nov. 1948, pp. 193-204 (V. 45)

Standardized precast reinforced concrete structural members and panels point toward economy in fireproof construction of ordinary buildings up to four stories. The method described has been so developed that architects and engineers can select from tables reinforced concrete beams, columns, roof, floor, and wall panels in the same manner as for structural steel. Design is in accordance with the ACI Code and costs compare favorably with similar structures framed of steel and fireproofed with concrete.

SAND GRADING INFLUENCE ON AIR ENTRAINMENT IN CONCRETE.45-12

Price 35 cents.

M. A. CRAVEN—Nov. 1948, pp. 205-216 (V. 45)

Four series of mixes with varying air-entraining agents, cement factors, and sand grading and content were prepared in order to observe sand grading effect on concrete. Graphic record is presented of air content and W/C plotted against fineness modulus of sand, flow and compressive strength plotted against fineness modulus of sand, and air content plotted against percent of No. 30—No. 50 sand and total sand percentage. Generally the percentage of air entrained in concrete increased with decrease in fineness modulus of sand. Quantity of air appears to be a function of the quantity of No. 30—No. 50 sand. Effect of grading and quantity of sand on other properties of fresh and hardened concrete is noted.

INFLUENCE OF SIZE GRADING OF SAND ON AIR ENTRAINMENT.45-13

Price 35 cents.

E. W. SCRIPTURE, Jr., F. B. HORNIBROOK, and D. E. BRYANT—Nov. 1948, pp. 217-228 (V. 45)

Field reports indicated difficulty with certain sands in securing the desired amount of entrained air, this was frequently attributed to the size grading of the sand, particularly to a deficiency in the finer fractions. An experimental investigation was undertaken in view of the paucity of published data on this subject. Mixes of sand and water alone, 1:4 and 1:2 mortars, and concrete mixes were made with and without air-entraining agents and the air contents determined. While size grading of the sand had a very large influence on air entrainment in a mixture of sand and water alone, this effect was smaller in mortars and very small in concrete mixes. It was found that maximum air was entrained by the 28–48 mesh size sand rather than the 48–100 mesh size.

USE OF PREPACKED AGGREGATE CONCRETE IN MAJOR DAM CONSTRUCTION.....45-14

Price 35 cents.

PAUL BAUMANN—Nov. 1948, pp. 229-236 (V. 45)

This paper sets forth the method of construction of a major gravity dam whereby the average temperature of the mass concrete may be controlled within the mean annual and the dry shrinkage reduced to a fraction of that of conventional concrete by means of prepacking of the coarse aggregate to a density corresponding to 30 percent voids, precooling the aggregate, the intrusion of the coarse aggregate by mortar with a cement content of two sacks per cu yd or less, and the internal post-cooling, curing and draining of the mass concrete. (This process is known as the Prepakt method of construction.)

RATIONAL ANALYSIS AND DESIGN OF TWO-WAY CONCRETE SLABS.....45-15

Price 60 cents.

C. P. SIESS and N. M. NEWMARK—Dec. 1948, pp. 272-316 (V. 45)

A new method for the design of two-way building slabs is proposed and its step-by-step development is described. A new moment distribution procedure is used to compute moments in a number of rectangular slabs continuous over rigid beams. Several variables are studied in the analyses and include: the ratio of sides, the effect of discontinuous edges, the torsional stiffness of the beam, various types of loading, and combinations of panels of various sizes and shapes. Certain conclusions are stated regarding the types of loading to be considered and the values of beam torsional stiffness to be assumed in the development of the design procedure.

Moments obtained in the foregoing analyses are modified to take into account the effect of additional variables. First, the slab moments are increased by various amounts as a result of the deflection of the beams. Next, separate coefficients for dead and live load are replaced by combined coefficients for a live load-dead load ratio of 3.0. Then follows a study to determine the effect of discontinuous edges, and this variable is eliminated from the procedure. Finally all slab moments are reduced by 20 percent in recognition of the redistribution of moments at high load. Additional studies are then made of the distribution of moments across the width of the slab and of the moments to be used for the design of the beams to complete the development of the design procedure.

A proposed design procedure is presented in specification form and is compared with other procedures of a similar nature and with the moment distribution procedure described in the appendix.

CORROSION RESISTANCE TESTS OF CONCRETE FLOORS—WITH AND WITHOUT METALLIC AGGREGATE.....45-16

Price 35 cents.

W. SCRIPTURE, JR. and C. H. SAKRYD—Dec. 1948 pp. 317-324 (V. 45)

An attempt has been made to determine the conditions under which concrete floors can be expected to give good service. Resistance to abrasion was determined on specimens with and without metallic aggregate after exposure to typical reagents selected from the groups described by the Joint Committee as having corrosive effects of an intermediate nature between no attack and severe attack. On the basis of the experimental results the reagents in the Joint Committee report have been classified in relation to anticipated service of concrete floors.

FLEXICORE PRECAST FLOOR AND ROOF SLABS.....45-17

Price 35 cents.

GAYLE B. PRICE—Dec. 1948, pp. 325-340 (V. 45).

This paper describes briefly the design, manufacture, erection procedures and use of Flexicore slabs in precast concrete floor and roof systems. The Flexicore slab has a standard cross section 12 in. wide and 6 in. thick, lightened by two holes 4⅝ in. in diameter running the full length of the slab which is made in inch variations of length up to a maximum of 22 ft 6 in. Reinforcement consists of two bars in the top of the slab for "handling" stresses and two or three prestressed tension bars at the bottom to take care of bending moment.

PROPOSED RECOMMENDED PRACTICE FOR THE APPLICATION OF PORTLAND CEMENT PAINT TO CONCRETE SURFACES.....45-18

Price 35 cents.

REPORT of COMMITTEE 616—Jan. 1949, pp. 353-368, (V. 45)

This report is a studied revision by a reorganized committee of the previous report of Committee 616 published in June 1942, based upon criticisms of that report plus information from more recent tests and experiences. Included are recommendations for appropriate usage, age of concrete, preparation of surface, and the preparation, application and curing of portland cement paint. Three appendixes discuss composition, manufacture and storage and, general characteristics and factors affecting durability.

DRY MORTAR AS A BEARING AND GROUTING MATERIAL.....45-19

Price 35 cents.

BOYD S. BROOKS—Jan. 1949, pp. 369-380, (V. 45)

In 1939, at the navy's new David Taylor Model Basin at Carderock, Md., there was an urgent need for a non-shrinking bearing material to be used under the cast-steel "chairs" which support the rails on which the towing carriages run. Requests for information concerning materials and installation procedures produced such varied answers that investigation was started to find the best material and the best procedures for placing it.

The bearing material selected for use was 1:3 portland cement mortar, with 4.13 gallons of water per sack of cement. This was rammed firmly into place.

This bearing material has now been in place over a period of about 9 years under extremely severe conditions and without exception has proved to be excellent in function and condition.

DIRECT DIMENSIONING OF RECTANGULAR SECTIONS.....45-20

Price 35 cents.

MICHEL BAKHOUM—Jan. 1949, pp. 381-396 (V. 45)

On the basis of the standard theory for reinforced concrete design, methods have been derived for the direct dimensioning of rectangular sections, with or without compression reinforcement, when subjected to simple bending, eccentric compression or eccentric tension. Two cases are considered: when the depth of the section is fixed and when it is not fixed. The conditions leading to minimum reinforcement are also studied. The solutions developed have been simplified by the use of curves which are applicable for pure bending as well as for eccentric compressive or tensile forces and cover any values of f_c, f_s and n. The curves allow making a variety of designs in a relatively short time and thus help in choosing the most economical or most suitable section.

STRENGTHENING BRIDGE SLABS WITH GROUTED REINFORCEMENT.45-21

Price 35 cents.

S. O. ASPLUND—Jan. 1949, pp. 397-408

On the project described the negative reinforcing bars settled as much as 2½ in. out of correct position. Various means for incorporating negative reinforcement at correct height in the finished structure are discussed. The method selected of grouting additional bars in grooves cut with the aid of diamond saw is described. Tests on beam specimens made with some bars encased in the concrete and some bars grouted into diamond-sawed grooves, give results identical with those to be expected for all bars normally encased; ultimate strengths conform closely to the plastic bending capacity of reinforced concrete according to design methods originated by C. S. Whitney.

COMPARATIVE DESIGNS OF A SEGMENTAL SKEWED FRAME CONCRETE BRIDGE BY THE STRAIGHT LINE AND PLASTIC THEORY METHODS.....45-22

Price 35 cents.

MILTON BRUMER—Jan. 1949, pp. 409-420 (V. 45)

The designs of a segmental skewed frame concrete bridge are discussed and compared as developed by both the straight line and plastic theory methods. The plastic theory will lead to rigid concrete frames of more economical proportions capable of sustaining equal or greater loads than frames designed by the straight line method. Effects of volume changes and yielding supports are also of smaller consequence. Deflections may be expected to be greater. The author concludes that further substantiation of the plastic theory is desirable.

SOME FACTORS AFFECTING AIR ENTRAINMENT.....45-23

Price 35 cents.

E. W. SCRIPTURE, Jr. and F. J. LITWINOWICZ—Feb. 1949, pp. 433-444 (V. 45).

A program was undertaken to investigate the influence of various factors on the amount of air entrained in concrete mixes with and without air-entraining agent added. The variables so far studied are slump, cement factor, and sand-total aggregate ratio. The results indicate that the two latter factors are of major importance but that slump has less effect. In general, entrained air content increases with increasing slump, decreases with increasing cement factor, sand-total aggregate ratio.

EXPERIMENTAL AIDS IN STRUCTURAL CONCRETE DESIGN...45-24

Price 35 cents.

R. E. GLOVER, O. J. OLSEN and CARL ZANGAR—Feb. 1949, pp. 445-468 (V. 45).

Statically determinate and indeterminate structures are generally designed by analytical methods. This paper describes several experimental aids which have been used in the design and stress analysis of statically indeterminate structures at the laboratories of the Bureau of Reclamation. These methods are used where analytical procedures cannot conveniently be applied. Described herein are photoelastic methods using both the polariscope and interferometer, the Beggs Deformeter, Stresscoat and electric SR-4 strain gages. Applications are given for each method.

PRACTICES, EXPERIENCES, AND TESTS WITH AIR-ENTRAINING AGENTS IN MAKING DURABLE CONCRETE.....45-25

Price 35 cents.

R. F. BLANKS and W. A. CORDON—Feb. 1949, pp. 469-488 (V. 45)

The authors discuss the advantages of air entrainment with regard to durability, permeability, workability, reduction in alkali expansion, time saving, reduction in water and cement, temperature rise, strength, abrasion resistance, monolithic lightweight concrete and mass concrete. U. S. Bureau of Reclamation experiences are discussed with regard to pumping, transportation and placing, grading of aggregates, factors affecting the amount of air entrained Air-entraining agents and air-entraining cements are discussed briefly. There is a brief section on the measurements of entrained air in concrete with a recommended procedure.

CONCRETE FLOOR FINISHING.....45-26

Price 35 cents.

GERALD MILSOM—Feb. 1949, pp. 489-492 (V. 45).

A concise discussion of the requirements for durable concrete floors including careful selection and control of materials and attention to weather conditions. The importance of proper equipment, working conditions and curing of the finished concrete are emphasized.

PLASTIC FLOW OF CONCRETE AT HIGH OVERLOAD.....45-27

Price 35 cents.

J. R. SHANK—Feb. 1949, pp. 493-500 (V. 45).

New data on the plastic flow of concrete prompted a reconsideration of the assumption that plastic flow is directly proportional to the unit stress applied. Tests are described in which deformations of 6 x 12-in. concrete cylinders were measured while under sustained loads approaching the ultimate strength. Plotted data are shown to demonstrate that the change in the rate of plastic flow appears to be uniform up to three-quarters or more of the ultimate strength with an abrupt change at the "true" ultimate strength. Included are other data indicating that the plastic flow curve has the same general form independent of time and applied load leading to a new consideration in expressing factor of safety and developing beam theories on a factor of safety basis.

CONTROL OF CONCRETE PAVEMENT SCALING CAUSED BY CHLORIDE SALTS.....45-28

Price 35 cents.

B. D. TALLAMY—Mar. 1949, pp. 513-520 (V. 45)

Under modern traffic requirements on heavily traveled roads salt-treated abrasives will not remove ice quickly enough. As maintenance forces have struggled to meet the demand for uninterrupted service the use of straight salts has become increasingly common. Direct applications of up to 600 lb of salt per two-lane mile are required to combat extreme icing conditions. Pavements constructed to withstand the weak brines deposited by salt-treated abrasives cannot stand up under straight salt. New pavements appear more vulnerable than those two or more years old.

The observed resistance to salt action of the oil soaked center streak of uphill traffic lanes led New York state highway engineers to investigate the feasibility of the use of dilute oil applications as a protective measure. Laboratory tests indicate successful results may be expected. In the late summer of 1948 oil spray equipment was developed in time to treat nearly 60 miles of new concrete highways, which should provide a wide base for field observation of the effectiveness of the method.

EXPERIENCE WITH AIR-ENTRAINING CONCRETE IN NEW JERSEY...45-29

Price 35 cents.

CHARLES M. NOBLE—Mar. 1949, pp. 521-528 (V. 45)

This paper outlines experience in New Jersey with air-entraining concrete utilized primarily to combat attack by de-icing chemicals. Damage to concrete pavements in New Jersey usually is associated with heavy loads. Freedom from disintegration troubles, except in isolated cases, is attributed to excellent aggregates, rigid laboratory control, mix design, inspection of material and plant equipment and construction supervision at the jobsite.

Air-entraining portland cement concrete was first specified for a concrete pavement contract in 1945 and since then has been used on many contracts with notable success to prevent deterioration due to scaling. The same high standards for materials, mixture design, laboratory control, field inspection and attention to details must be maintained as with ordinary concrete. Pavements thus far constructed in New Jersey with air entrainment have not

scaled or shown any signs of disintegration when ice control chemicals have been properly mixed with abrasives at a rate not exceeding 75 lb per cu yd of abrasive. It is too early to form judgment but indications are the results achieved fully justify the use of air-entraining concrete.

DESIGN DETAILS FOR ARCHITECTURAL CONCRETE.....45-30

Price 35 cents.

J. J. HOGAN—Mar. 1949, pp. 529-540 (V. 45)

"To achieve beauty with concrete, as with any other material, requires studied design and skillful craftsmanship" are the words of architect Robert D. Murray. This paper directs attention to the matter of "studied design". The skilled craftsmanship required to construct architectural concrete must be preceded by proper design by the architect and engineer. The designer must understand and appreciate the fundamental requirements of this type of construction. This paper discusses considerations appropriate to achieving the desired texture of surface as affected by form materials selected. Ornamentation should be considered in every detail from the standpoint of practicability. Construction joints are inevitable and should be specified in logical locations. Similarly, control joints should be specified with an understanding of where the most likely cracks due to volume change will occur and the design of reinforcement should be coordinated with the location of the control joints. Numerous illustrations support the text.

CONSTRUCTION PRACTICES FOR ARCHITECTURAL CONCRETE.....45-31

Price 35 cents.

E. B. OBERLY—Mar. 1949, pp. 541-552 (V. 45)

Detailed plans and carefully drawn specifications executed by a contractor and superintendent with the know-how of form construction are necessary for good architectural concrete. This paper describes some of the more important construction practices to be considered to achieve the results desired. Form details and concrete quality are discussed in considerable detail, the former being supported by illustrations.

CONCRETE OF THE FUTURE 45-32

Price 35 cents.

ROBERT F. BLANKS—April 1949, pp. 565-568, (V. 45)

Mr. Blanks in his remarks as retiring president, depicts what concrete might be 50 years in the future and discusses the role of the engineer in that era.

CONTROL OF QUALITY OF READY MIXED CONCRETE 45-33

Price 35 cents.

STANTON WALKER—Apr. 1949, pp. 569-580 (V. 45)

Control of quality of ready-mixed concrete is discussed. The objective of the paper is to pose the more important problems of concrete control, especially those peculiar to ready-mixed concrete, with the view of eliciting discussion, rather than to attempt to outline recommended practices. Both central-mixing and transit-mixing are discussed. Attention is directed to the mutuality of responsibility of the producers and users in bringing about effective control.

TESTS OF LIGHTWEIGHT-AGGREGATE CONCRETE DESIGNED FOR MONOLITHIC CONSTRUCTION....45-34

Price 35 cents.

WALTER H. PRICE and WILLIAM A. CORDON—Apr. 1949, pp. 581-600 (V. 45)

The characteristics and origin of lightweight aggregates are described and the comparative results of laboratory tests of strength, insulating value, shrinkage, and weathering resistance of concrete made with light weight aggregates from 17 producers are summarized. Data are included on expanded shale, expanded slag, scoria, pumice perlite, exfoliated vermiculite and diatoamceous earth.

WHAT HAVE WE LEARNED ABOUT AIR-ENTRAINING CONCRETE?.....45-35

Price 35 cents.

J. F. BARBEE—Apr. 1949, pp. 601-612 (V. 45)

A review of ACI published data on air-entraining concrete, this paper summarizes methods of control and the effect of air entrainment on durability, plasticity and workability, strength, resistance to abrasion and other properties of concrete. The effects of variations in cement and water content, sand-coarse aggregate ratio, mixing time and the use of calcium chloride as an admixture are described. Changes in technique necessary to successfully place, vibrate and finish air-entraining concrete are pointed out. A detailed bibliography follows.

DEVELOPMENTS IN THE MANUFACTURE AND TECHNOLOGY OF CONCRETE MASONRY UNITS.....45-36

Price 35 cents.

JAY C. EHLE—Apr. 1949, pp. 613-620 (V. 45)

The author touches the high spots of developments in the manufacture of concrete masonry units listing the many problems which now face the manufacturer. Rapid growth of the industry in the past ten years and the change from tamping to vibration methods is described. New plants are notable for labor saving equipment compared with those of the middle 1930's. In addition to block manufacture and several methods of curing, in use and proposed, the author mentions briefly: block-handling equipment, maintenance and repair of the plant, and the greater supervisory skill now required for successful operation.

LIGHTWEIGHT-AGGREGATE CONCRETE.....45-37

Price 35 cents.

RALPH W. KLUGE, MORRIS M. SPARKS, and EDWARD C. TUMA—May 1949, pp. 625-644 (V. 45)

Results of studies on lightweight-aggregate concretes made by the National Bureau of Standards at the request of the Housing and Home Finance Agency (formerly the National Housing Agency) are reported in this paper. Aggregates included were expanded clay, shale, and slate; three types of expanded blast-furnace slag; expanded vermiculite and perlite; sintered diatomite and fly ash; and a pumice. Concrete of various proportions were made from these aggregates and determinations were made of the weight per cubic foot, compressive and transverse strength, shrinkage, elastic modulus, absorption, resistance to freezing and thawing and thermal conductivity. Test results are briefly discussed and summarized.

ANALYSIS OF BUILDING FRAMES WITH UNSYMMETRICAL DIFFERENTIAL SETTLEMENT OF THE FOUNDATIONS....45-38

Price 35 cents.

G. A. LEONARDS—May 1949, pp. 645-652, (V. 45)

The rational design of building frames founded on soil requires a consideration of the effect of settlements caused by the building loads. This paper discusses a procedure for estimating the stresses in such frames due to unsymmetrical differential settlement of the foundations. The procedure is based on the slope-deflection method for analyzing rigid frames, but adapts it to permit a relatively rapid solution by the use of successive approximations.

EFFECTS OF MIXING TIME, SIZE OF BATCH AND BRAND OF CEMENT ON AIR ENTRAINMENT ...45-39

Price 35 cents.

E. W. SCRIPTURE, Jr., and F. J. LITWINOWICZ—May 1949, pp. 653-664, (V. 45)

A program, partially described in previous papers, to investigate various factors affecting the amount of air entrained in concrete has been continued. The factors covered in this paper are mixing time, size of batch, and brand of cement. The entrained air rises to a maximum in the early stages of mixing and thereafter decreases. Provided mixing is adequate, the size of batch does not appear to affect the amount of air entrained. Wide variations are found in the amounts of air entrained by different cements and it does not appear possible to correlate these variations with any easily and quickly determinable characteristic of the cement.

A METHOD FOR DETERMINING THE AIR CONTENT OF FRESH AND HARDENED CONCRETE 45-40

Price 35 cents.

ROBERT P. VELLINES and THOMAS ASON—May 1949, pp. 665-672, (V. 45)

A method and equipment are described for determining the air content of fresh and hardened concrete based on the principle that air entrained in concrete reacts to external pressures and that volume changes follow the gas laws. The air entrained in hardened concrete reacts to an external pressure in the same manner as an equal volume of air would if it were not surrounded by hardened cement paste. The volume of the air in the specimen is proportional to the time required for a quantity of gas to escape through an orifice from a pressure of p_1 to a pressure of p_2 at a constant temperature.

STUDY OF DUSTY CONCRETE CEILINGS 45-41

Price 35 cents.

JACOB FELD—May 1949, pp. 673-680, (V. 45)

In a large Eastern housing project of two-story concrete buildings considerable trouble was experienced from the "dusting" of first floor ceilings which were poured against new pressed-board forms. The experimental work to isolate the cause, the results of the tests and their interpretation indicated that the dusting was caused by an acidic resinous substance in the surface of the form board. The substance on the surface of the board appeared to be neutralized when the form was used for the first time, inasmuch as reused form boards gave little trouble. In concluding, the author recommends that new types of form material be checked for possible chemical reaction before use in large amounts.

AN INVESTIGATION OF HYDRATING CEMENTS AND RELATED HYDROUS SOLIDS BY DIFFERENTIAL THERMAL ANALYSIS 45-42

Price 35 cents.

GEORGE L. KALOUSEK, CURTIS W. DAVIS, Jr., and WILLIAM E. SCHMERTZ—June 1949, pp. 693-712 (V. 45)

Differential thermal analysis was used to identify some of the hydrous solids formed in cements and to follow partially the course of some of the hydration reactions. Pastes of nine commercial portland cement clinkers with and without additions of gypsum were tested at 1, 3, 7, 28 and 90 days and 9 months. Among the individual solids prepared and studied were the calcium sulfoaluminates, hydrous calcium aluminates, lime-silica gels made from different starting materials, with and without additions of Al_2O_3, SO_3 and Na_2O. Thermal analysis results show that the isometric tricalcium aluminate hexahydrate does not form in hydrating cements. The trisulfate form of the calcium sulfoaluminate and the solid solution, $3CaO{\cdot}Al_2O_3{\cdot}CaSO_4{\cdot}13H_2O$-$3CaO{\cdot}Al_2O_3$ $Ca(OH_2)$-$12H_2O$, appear as metastable products only. The stable sulfate-bearing phase has not yet been isolated but presumably is characterized in thermal analysis by an endothermic bulge at 130-140 C. A possible role of Fe_2O_3 in the formation of this new phase is considered. The lime-silica gels may occur in two forms, one exhibiting two distinctive thermal changes and the other not showing any marked changes; the former is the more unstable of the two, transforming in prolonged tests to the latter. Alumina appears to stabilize the less stable form of these gels.

STRESSES IN REINFORCED CONCRETE DUE TO VOLUME CHANGE.45-43

Price 35 cents.

FRANK R. BEYER—June 1949, pp. 713-724 (V. 45)

A study of stresses set up in reinforcing steel and concrete by volume changes. SR-4 gages were attached to the reinforcing steel and carefully waterproofed. The test specimens were poured in essentially watertight forms. Strain readings began immediately and continued at varying intervals for three months. A drift test to determine the effect of diminishing battery potential was included to correct for this variable. Charts are presented to show the stresses measured during the tests. The author concludes that with proper precautions the SR-4 strain gage will measure the actual stress induced by volume changes. These stresses, occurring in reinforced concrete in a definite cycle with respect to time, are appreciable and should be considered in design.

MONOLITHIC AND BONDED FLOOR FINISHES.....45-44

Price 35 cents.

MORGAN B. KLOCK—June 1949, pp. 725-732 (V. 45)

The author discusses the design and construction of two types of floor finish. A floor capable of withstanding heavy wear, either bonded or monolithic finish, can be obtained by using a stiff mix of sound, well-graded materials which is thoroughly compacted with a power float. Hard coarse and fine aggregates are recommended for the greatest resistance to wear.

PUMPED CONCRETE FOR ENDERS DAM SPILLWAY.....45-45

Price 35 cents.

C. O. CRANE—June 1949, pp. 733-740

A detailed description is given of the use of pumped concrete in the construction of the concrete spillway for Enders Dam. The author calls attention to the many features of pumped concrete that are of interest and value in considering this method of concrete placement.

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